WAGE WORKS JOURNAL

VOL. I MAY, 1933 No. 1

Special Articles

1726 this pipe was laid -still in service 1935

THE cast iron water main shown above has served the German city of Ehrenbreitstein for more than two centuries. Over his official seal, the Burgomaster recently wrote, "The water main . . . was replaced in 1725 by cast iron pipes . . . across the much-frequented tramway . . . and lies partly under the pavement along the Anrather with its heavy traffic. On the occasion of an excavation the enclosed photographs have been made . . . these show the position and the good condition of the pipes." Note that the pipe has bell-and-spigot joints—it is but one of many installations which demonstrate that the useful life of cast iron pipe is more than a century.

The four major economies resulting from the long life of cast iron pipe are due to its effective resistance to rust. Cast iron is the only ferrous metal for water and gas service that [illegible] rust. This characteristic makes [illegible]

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For further information, address The Cast Iron Pipe Research Association, Thomas F. Wolfe, Research Engineer, 1015 Peoples Gas Building, Chicago, Illinois.

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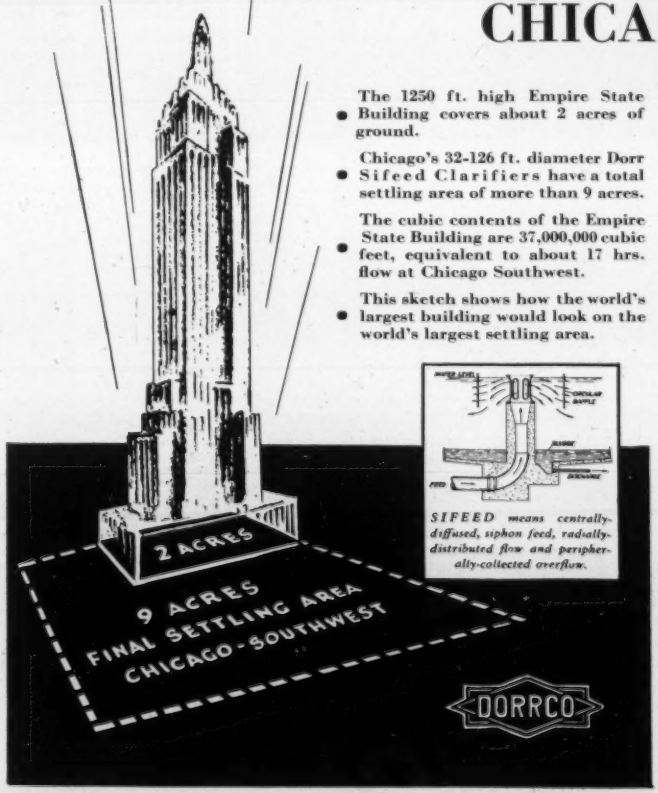

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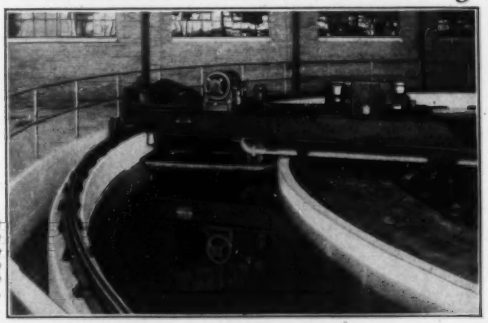

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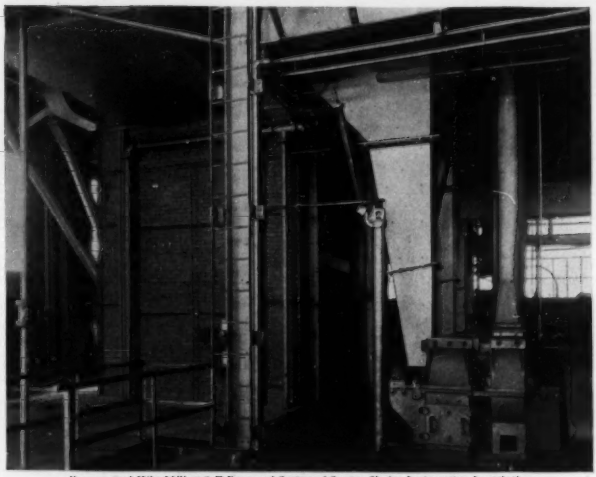

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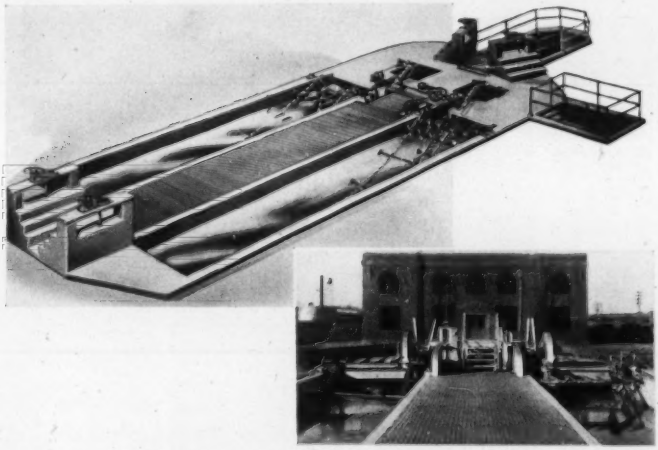

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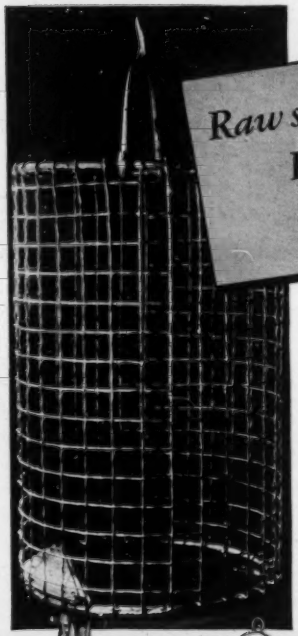

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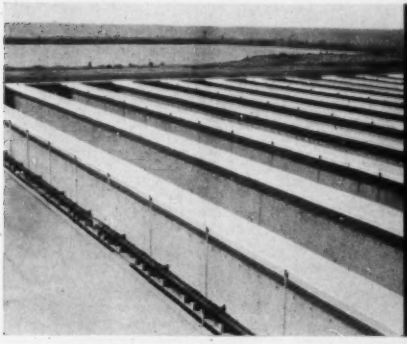

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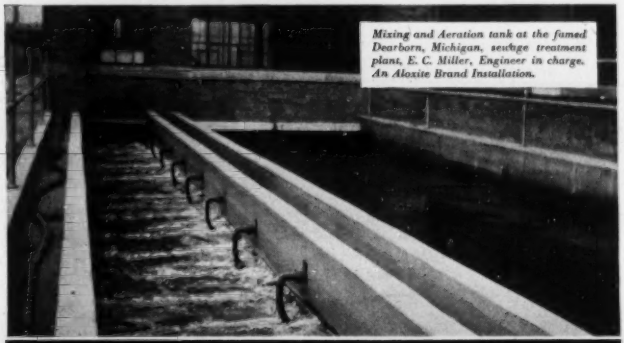

Mixing and Aeration tank at the famed Dearborn, Michigan, sewage treatment plant, E. C. Miller, Engineer in charge. An Aloxite Brand Installation.

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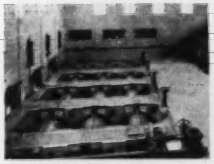
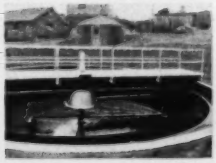
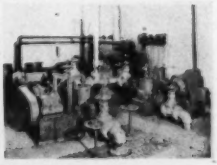
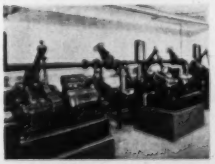
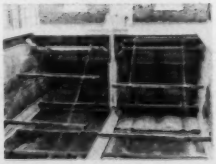




In Sewage Disposal and Water Treatment Plants

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4

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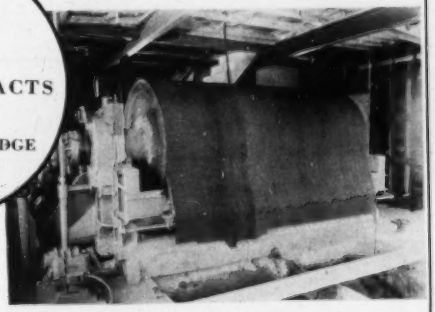
DEWATERING



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Sewage Research

STUDIES OF SEWAGE TREATMENT. III. THE CLARIFICATION OF SEWAGE. A REVIEW*

By Emery J. Theriault

Principal Chemist, U. S. Public Health Service

It is frequently remarked that present day methods of sewage treatment have greatly outstripped the development of rational theories of plant behavior. This situation is by no means unique in the general field of practical applications. In sewage treatment, nevertheless, the rationalization of plant procedures has doubtlessly been greatly retarded by the, at times, hopeless outlook of the fundamental problem regarding the quantitative description of reactions between the oxygen-consuming impurities of sewage, at a concentration of approximately 0.001 M, and the various kinds of "activated sludges," also known as biological slimes and more accurately described as zoogleal aggregates.

With such unpromising material at hand, it is to be expected that the theories in vogue are generally founded on more or less pertinent analogies with more amenable systems rather than on the direct examination of sewage colloids or bio-colloids. Sustained research, for example, on adsorption as a mechanism for the removal of organic matters from sewage by sludges becomes impossible when no adsorption is observed, using sludge drawn from an activated sludge plant. Likewise, the evaluation of various theories of colloidal precipitation is a discouraging task when the sewage colloids refuse to agglomerate or when the agglomerated solids decline to settle. The development of biological theories is greatly hampered by unsurmounted difficulties in the matter of cultivating protozoa and allied organisms in pure culture; bacterial counts may fail to yield more than 1 per cent of the organisms present in the sludge. Theories of enzymatic action have frequently been advanced but in no case have the hypothetical clotting, lytic or oxidative agents been produced.

In another sense, the present unsatisfactory state of the sewage problem is closely related to the controversy, as reviewed by Wilson (1930),

* This is the third of a series of papers summarizing studies of the activated sludge process, conducted at the Stream Pollution Investigation Station, United States Public Health Service, Cincinnati, Ohio.

"over the imagined respective functions of the biological and physical agencies." No single theory has been proposed that will account satisfactorily for all of the diverse phenomena of sewage clarification and stabilization. In general, the physical or colloidal theories have explained the purification either by the coagulation of the sewage colloids or by the absorption of the organic matters on the sludge, to the frequent neglect of the subsequent biological oxidation of the coagulated or absorbed impurities. On the other hand, this preliminary process of clarification has often been ignored or even denied by the proponents of the biological theories.

Biological oxidation in some form must necessarily be considered in any general theory of sewage purification by the usual processes. Greater uncertainty exists as to the validity of the clarification concept. For the purposes of the present discussion, leading to the presentation of a new theory of sewage clarification in a series of papers, the review of the scattered literature will therefore be devoted largely to the phenomenon of clarification as a process preliminary to and distinctly separated from the subsequent process of biological oxidation.

Earlier Theories

The earliest recorded scientific investigation of sewage treatment is probably that of Frankland (1868) with the British Rivers Pollution Board. Working under laboratory conditions, with glass cylinders, Frankland gave the first clear-cut demonstration of the role of atmospheric oxygen in the purification of sewage and laid the foundation for the principle of "intermittent filtration." Frankland assumed that the dissolved organic matters of sewage were oxidized by the oxygen of the air as the sewage dripped through a filter or a porous soil.

With the recognition of the biological character of filter growths, the simple theory of direct oxidation advanced by Frankland became untenable. The numerous reports of the Lawrence Experiment Station (1887 et seq.) are to be consulted for the development of theories of direct biological oxidation of organic matters in the *slow* passage of the sewage through a filter. Commenting on these earlier observations Dunbar (1908, pp. 121–2) says: "If Frankland's experiments are to be regarded as the basis of modern biological methods of sewage treatment, they have really attained this position with the aid of the systematic and scientifically conducted experiments of the Massachusetts State Board of Health."

Dunbar's classical experiments were conducted at the Hamburg Hygienic Institute (1897–1900). He considers that the oxidation of sewage matters in their slow passage through a filter was a possible explanation ". . . so long as it could be maintained that sewage remained

about three days in the filter; but when it was shown that the sewage left a 3-foot filter thoroughly purified within ten minutes—the above explanation fell to the ground. It cannot be assumed that micro-organisms decompose highly complex molecules of organic substances within a few minutes or seconds. . . . It can only be assumed that the dissolved organic matters are first separated from the sewage during its passage through the filter, and are retained in the filter to be decomposed and oxidized by the micro-organisms during the succeeding period of rest." After examining theories of mechanical retention and of chemical combination Dunbar concludes that "an explanation of the purification process must be sought in absorption phenomena."

The general theory of filter action proposed by Dunbar, and systematized by Ardern (1921), was somewhat as follows:

1. Coarse particles are transferred to the filtering media by surface attraction.

2. Dissolved matters are retained by adsorption phenomena accelerated by the growth of a biological slime on the filter materials.

3. The absorbed materials are oxidized by biological action and also by oxygen "ozonized by the high pressures existing in the gelatinous film."

4. The absorption process is maintained by the action of micro-organisms under aerobic conditions.

5. There is a residue of organic matter, highly resistant to bacterial action, which either accumulates in the filter or is discharged with the effluent.

Dunbar's concept of the action of sewage filters is therefore properly described not simply as an "Absorption Theory" but rather as a well-balanced summary of the then existing knowledge regarding sewage purification.

Stoddart's criticism of Dunbar's theory will serve as an illustration of objections brought to bear against absorption as a step preliminary to biological oxidation. "It has already been shown that the starting point is the assumption that micro-organisms cannot 'decompose highly complex molecular organic substances within a few minutes or seconds' . . . and with the exposure of the fallacious nature of this assumption, which may be characterized as an unwarrantable repression of scientific imagination, the theory itself falls to the ground" (Stoddart, 1909, p. 201). Stoddart's experiments controverting Dunbar's absorption theory were based on comparisons of rates of flow through filters of solutions of sodium chloride and of ammonium chloride. It will be shown in subsequent papers of this series that the adsorption of ammonium chloride should be prevented by the sodium chloride, unless special precautions were observed. It is also possible that the adsorptive capacity of Stod-

dart's filter was exceeded. It must be admitted, nevertheless, that Dunbar's theory when first proposed was by no means free from objections, particularly in the matter of the removal from sewage of dialyzable or non-colloidal matters such as ammonia.

Biltz and Kröhnke (1904), in their important paper on the first definite recognition of the colloidal nature of sewage, list three methods whereby the clarification of sewage may be accomplished. Chemical precipitation is represented by the use of ferric salts. Biological methods of sewage treatment are regarded as affording a means for the formation of absorption compounds between the colloids of sewage and the slimy, gelatinous coatings of the filter materials. These biological slimes, however, are regarded simply as colloid surfaces. The bacteria themselves, although identified with the biological slimes, are regarded as precipitants in reversed analogy to agglutination. The mechanical theory proposed by Biltz and Kröhnke is referred by them to the absorption theory of Dunbar and earlier writers; they do claim, however, to have established a rational connection between the nature of sewage and the mechanism of clarification. As to the fate of the absorbed materials, it was assumed that they might be oxidized by the action of "ferments" or else directly (unmittelbar) by atmospheric oxygen, "at all events essentially by purely chemical means" (cf. Jones and Travis, 1906, pp. 72 and 161). This unsubstantiated part of the theory advanced by Biltz and Kröhnke does not detract greatly from their demonstration that at least one-third to one-half of the sewage matters are present as colloids.

Jones and Travis (1906), in their widely discussed paper on the clarification of sewage by filters at Hampton (England), advanced a somewhat different theory for the "de-solution" of sewage colloids. They maintained that the deposition of solids on the filter materials "is dependent upon the action of surfaces, as such," and not on the colloidal properties of the biological slimes. The efficiency of a filter should therefore be increased by the use of finely divided material. Jones and Travis (1906, p. 192, discussion) were afterwards willing to concede to Biltz and Kröhnke, and also to H. W. Clark of the Lawrence Experiment Station, that ". . . colloids in gel form encouraged further deposition, and to this extent the bacterial coating of the (filter) material undoubtedly played a part."

Biochemical oxidation was practically disregarded by Jones and Travis (1906) and, as pointed out by Wilson (1930), could properly be omitted if their theory was restricted to sewage clarification. The claim that "the bacteria play only a subsidiary part in the purification" was, nevertheless, highly objectionable to proponents of biological theories. The view of bacterial purification held by Jones and Travis was clearly an expectancy of sludge liquefaction. "In Hampton, at the outset,

absolute confidence was reposed in voracious capacity of the bacteria. Indeed, the announcement was made that the solids of sewage would all be 'eaten by the organisms,'" thereby preventing the clogging of a relatively fine (0.5 inch) filter. Disillusionment in respect to the hydrolysis of the highly resistant sludge humus by micro-organisms is caustically expressed in the quoted statement "The best organism I have at the sewage-works is the man with a barrow" (cf. Dunbar).

Although the so-called "Hampton Doctrine" of Jones and Travis is frequently contrasted with the more general theory of Dunbar, it is clear that the views expressed by Jones and Travis, and also by Biltz and Kröhnke, are in substantial agreement as to the mechanism for the primary removal of suspended or colloidal matters from sewage. In each case a purely physical theory is proposed, which does not account for the removal of matters in true solution, such as ammonia, sugar, etc. The Dunbar theory, however, does make suitable provision for the subsequent oxidation of the absorbed materials. The introduction of the time element with its implication of preliminary absorption, instead of direct oxidation by atmospheric oxygen or direct action by bacterial cells, must be regarded as Dunbar's greatest contribution to the sewage problem.

Rate of Clarification

While it is plausible or even necessary to assume that some preliminary process of absorption must be operative in the clarification of sewage, it was by no means easy to separate this process from the accompanying process of biological oxidation when methods of sewage treatment were restricted to contact beds or trickling filters. The phenomenon of clarification may nevertheless be readily demonstrated by a suitable arrangement of stages in biological oxidizing devices utilizing "activated sludges" as absorbents. The experiments of Theriault and McNamee (1930) will serve as an example.

Using a biological device, designed to simulate a small stream with polluted water trickling over slime-covered stones, it was shown at the Stream Pollution Investigations Station of the U. S. Public Health Service that, starting with a grossly polluted water, the production of a highly clarified liquid could be effected in a period of flow of about 20 minutes. The apparent reduction in the incoming pollution, neglecting absorption, was of the order of 90 per cent. For polluted waters incubated under laboratory conditions, in the absence of deposited sludge, an equivalent degree of actual purification by biological oxidation could be accomplished only in 10 days, under comparable temperature conditions. The term "clarification" will be used in referring to the marked over-all improvement which results when sewage is treated for a brief period of time with certain biological slime-growths, the so-called

activated sludges. The term "purification" is properly restricted to the relatively slow but none the less effective process of biological oxidation. It is obviously desirable to distinguish clearly between these two methods whereby the pollutional characteristics of a sewage may be diminished.

Reference will again be made to these experiments in this paper. Their particular virtue lies in the fact that the time-element was very clearly defined and, also, that the tests were conducted on a semi-plant scale in close approximation to natural conditions. It is reasonable to assume that the phenomenon of clarification demonstrated in these experiments is identical with the rapid improvement noted in trickling filters, contact beds and similar devices where the control of experimental conditions offers many difficulties. In each case, however, it is certain that biologically active sludge is present and it can be assumed that the period of contact is brief, although divergent opinions on this particular point have been expressed.

Clarification, likewise, undoubtedly occurs in the activated sludge process, where the biological slimes are blown through the sewage instead of remaining adherent to stones as in processes of filtration. The time-element in the activated sludge process can only be estimated with difficulty in an actual installation. Activated sludge, nevertheless, is admirably adapted to laboratory experimentation and several workers have reported on its clarifying power under controlled conditions.

In the discussion following the paper by Young and Melling (1918, p. 19), Garfield refers to the experiments of Barraclough, in which a reduction of about 50 per cent in permanganate oxygen consumed, together with a decrease in free ammonia, was observed in absorption experiments where samples were taken "immediately" after mixing activated sludge with sewage. "The first point was that the activated sludge, upon being mixed with the sewage, immediately effected a purification which might be compared with the action of a very heavy chemical precipitant, ..." Barraclough evidently was dealing with a very "good" sludge.

Cambier (1920) presented experiments indicating that the removal of the colloidal matters of sewage, together with the ammonia, is effected from the very start of the contact period with activated sludge, although not at the first instant of contact as interpreted by Butler and Coste (1927). In work with fresh sludge approximately 50 per cent of the ammonia was absorbed in 15 minutes, without any corresponding indication of nitrification.

Butler and Coste (1927) were unable to confirm the claims of Cambier regarding the fixation of ammonia by activated sludge, although they did find a marked "at once" action of activated sludge upon sew-

age. The average improvement, measured in terms of permanganate oxygen consumed, was about 29 per cent when the period of contact was reduced to the minimum consistent with good mixing. "We think that the ability to produce a stable supernatant or filtrate as an immediate result of mixing activated sludge with sedimented sewage might constitute a test of activity of activated sludge."

Herb (1928) found that the oxidizability of the supernatant liquor was reduced by about 16 per cent when a sewage was aerated for 5 minutes in the presence of 25 per cent of activated sludge by volume. A steady state corresponding to 50 per cent reduction was reached in about 30 minutes.

Theriault and McNamee (1930) reported a reduction of about 65 per cent in the oxygen demand of a sewage following 10 minutes of agitation with a biological slime under laboratory conditions. This corresponds to the 90 per cent reduction in 20 minutes observed under plant conditions.

Parsons (1929) could not obtain more than a slight "instantaneous effect" in his sewage clarification experiments using activated sludge as an absorbent. He suggests that the results of Butler and Coste may be in error because of an intervening filtration. Parsons, nevertheless, did obtain a marked improvement in his equilibrium experiments using a 30-minute period of agitation.

Grant, Hurwitz and Mohlman (1930, p. 240) presents a very consistent set of absorption curves indicating a reduction of about 80 per cent in oxygen requirements in 40 minutes, with little change when the aeration of the sludge sewage mixtures was extended to 160 minutes.

Seiser (1928) concluded that the absorptive capacity of fully aerated activated sludge is largely exhausted in less than one hour.

The discrepancies in the graded series of observation just reviewed are probably more apparent than real. Negative results for the removal of ammonia, even though organic matters were readily removed, are satisfactorily explained by Butler and Coste (1927) on the basis that the sludge used in their experiments was drawn from a plant in which the second or nitrification stage of biological oxidation had not yet been fully established. As shown by complete deoxygenation curves, ammonia was not removed in the laboratory experiments of Theriault and McNamee (1930) using sludge drawn from the upper sections of their artificial stream. Ammonia was readily removed, however, at "downstream" sections of the same installation.

Values reported by different observers for the percentage removal of organic matter from a given sewage cannot, of course, be compared without reference to the strength of the sewage, the absorptive capacity of the sludge, and other factors. The "at once" effect noted by Bar-

raclough, and also by Butler and Coste, does not exclude later manifestations of the same phenomenon continued at a diminishing rate as the exhaustion of clarifying power is approached.

Negative results are seldom reported in the literature but they nevertheless have generally been obtained by the writer in preliminary experiments with sludges which are now believed to have been fully saturated with sewage matters. It should obviously be desirable to work with fully activated or regenerated material in any attempted demonstration of the clarification process. With these explanations, it must be concluded that the evidence regarding the existence of a very rapid process of clarification is reasonably definite. If the phenomenon in question does occur with a given sludge, it should be completed in 30 or 40 minutes when pushed to the limit of the absorptive capacity of the sludge.

It is necessary to keep this definition clearly in mind so as to distinguish between clarification, as such, and the subsequent reactivation of the sludge or the regeneration of its clarifying power through the biological oxidation of absorbed materials. The statement that "two or three hours are required for clarification by the activated sludge process" should be interpreted as including time for the period of regeneration. Clarification in the sense of turbidity removal, as by bacteria-eating plankton, is yet another phenomenon which is probably never completed in 30 or 40 minutes. On the other hand, the clarifying action of chemical precipitants may be regarded as closely related to the phenomenon under consideration. At all events the term "clarification" will be used only in the relative sense and not in the absolute sense with its connotation of complete removal of turbidity. A sewage may still remain quite turbid when clarified to the extent of 29 per cent as in the experiments of Butler and Coste.

Mechanism of Sewage Clarification

Having shown the relation of clarification to the general theory of sewage treatment and having presented the evidence regarding the validity of this useful concept, it will now be of interest to look more closely into some of the suggested explanations of the associated phenomena.

Mention has already been made of the explanation offered by Biltz and Kröhnke for the efficacy of ferric salts in the precipitation of sewage colloids which, on the basis of migration experiments, had been shown to be negatively charged. It is to be noted that these experiments refer to the sewage colloids and not to the sludge. Surface attraction was suggested by Dunbar, and also by Jones and Travis, as a mechanism for the removal of the coarser or suspended matters by filter materials or by deposited sludge.

Theories of mutual coagulation of sewage colloids by the neutralization of electrical charges have frequently been advanced, in distinction to precipitation by outside agencies. With due regard to conventions regarding electrical charges, Cavel (1931), for example, reports that negative colloids, such as Congo red, are adsorbed by activated sludge. Hence the sludge must be positively charged and, by further inference, the sewage colloids should be negatively charged. Dienert (1922) is less definite in his statement that the sewage colloids seem to be negatively charged while the sludge itself appears to be positively charged. Buswell (1928, p. 319) says, " One serious objection to the colloidal theory of coagulation is that the colloidal particles in sewage and the activated sludge particles are, so far as we are able to determine, both negatively charged. Since adsorption of colloids is most effective between oppositely charged particles it should not be applied to the conditions of the activated sludge particles without reservation." Baly (1931) considers that, were it not for this observation of Buswell and Long (1923), the most attractive theory concerning the mutual coagulation of sewage and sludge colloids by the neutralization of electrical charges, " would seem to be one that could offer a completely satisfactory explanation of the activated sludge and activated filter bed processes of purification." To avoid this difficulty Baly (1931) has proposed a theory in which, as reported by Lumb (1933), it is assumed " that weakly charged colloidal particles will associate with strongly charged bacteria of the same sign, because the free energy of the whole system is thereby decreased." It should be incorrect to conclude that, by one theory or another, sewage colloids will always be mutually coagulated, regardless of sign.

In criticism of the theories of sewage precipitation by colloidal phenomena it should be recalled that the suspended matters of sewage are by no means all of colloidal dimensions. Moreover, the colloidal theories at best cannot account for the removal of the considerable proportion of the nitrogenous and other oxygen-consuming impurities which are present in true solution. The presence of bacteria in the sludge particles should not be regarded as evidence of a transfer of the micro-organisms from the sewage to the sludge. Local growth on the favorable medium afforded by the sludge is to be considered. The actual removal of bacteria from the sewage liquor may be largely accomplished by bacteria-eating plankton. It is not clear, moreover, that the previous history of the sample has always been considered in determining the character of electrical charges on activated sludges. The pH value of the liquid is often disregarded and the magnitude of the electrical charge appears to have been generally ignored, except by Baly.

From another angle it must be considered that the Baly theory, as applied to sewage colloids, does offer an explanation for the agglomera-

tion of sewage colloids akin to the growth of drops by distillation from smaller to larger ones or, for that matter, to the growth of precipitates, as in water purification, from the stage of barely recognizable turbidity to the condition of visible "flocs." In this case there can be no question as to the phenomenon itself and, furthermore, it must be considered that all of the particles composing the flocs originally possessed the same electrical sign. It appears reasonable, therefore, to assume a similar mechanism for the agglomeration of sewage colloids into sludge particles. Separate consideration should be given to the absorption of other organic matters by the sludge particles.

The biological theories of clarification are so intermingled with notions of oxidation that mechanisms for the rapid removal of organic matters from sewage are frequently obscured, although Johnson (1914, pp. 130–1), with reference to the action of the upper portion of a filter receiving crude sewage is reasonably definite in his statement that "The filter material rapidly becomes coated with a slimy or gelatinous growth of Zoogloea ramigera, which may be regarded as a large number of bacteria embedded in a gelatinous matrix." The gelatinous character of these zoogleal formations enables them to "absorb *soluble* polluting substances—as already described in Dunbar's absorption theory." The functions of a different class of filter organisms is emphasized by Buswell (1928, p. 331): "As indicated in the previous discussion, there are two groups of reactions which take place in trickling filters, (a) bioprecipitation and (b) oxidation or nitrification, as the latter is usually called. Bioprecipitation is accomplished by the larger organisms, the chlamydobacteriaceae or 'giant bacteria,' the fungi ('especially molds'), the larger protozoa, and certain higher animals. These organisms are found to compose the bulk of the growths on the filter stones. A certain amount of oxidation is accomplished incidentally by these organisms since they give off carbon dioxide."

With particular reference to the activated sludge process, the "Bioprecipitation Theory" is described as follows: "In view of previous work of other authors cited and the data of the present paper, we wish to propose the following statement of the theory of the activated sludge process. Activated sludge flocs are composed of a *synthetic* gelatinous matrix similar to that of Nostoc or Merismopedia, in which filamentous and unicellular bacteria are embedded and on which various protozoa and some metazoa crawl and feed. The purification is accomplished by ingestion and assimilation by organisms of the organic matter in the sewage and its resynthesis into the living material of the flocs. This process changes organic matter from colloidal and dissolved states of dispersion to a state in which it will settle out" (Buswell and Long, 1923). Buswell (1931) describes activated sludge as follows: "The

animal inclusions of the sludge made up a very small part of the entire mass. The base of the sludge was composed of zoogleal masses intermixed largely with filamentous bacteria and occasional Zooglea ramigera." The surface area afforded by the zoogleal masses is estimated to be at least 500 square feet per cubic foot of aeration chamber. The importance of the zoogleal masses of bacteria has also been pointed out by Taylor (1930) and, more recently the cultural and other characteristics of these gelatinous formations have been studied by Butterfield (1935) in this laboratory.

Buswell (1928, p. 318–9) states that "there is practically no absorbed, precipitated or coagulated amorphous matter in these sludge particles, but that they are composed entirely of active, growing microscopic organisms. . . . From what we know of the metabolism of microorganisms it is probable that the unicellular forms absorb through their membrane such soluble forms of organic matter as are able to pass through this membrane, and that they also secrete enzymes which are capable of peptizing or liquefying colloidal particles too large to be directly absorbed. Protozoa, on the other hand, can easily be seen to approach and ingest visible particles of organic matter." The claim that activated sludge contains "practically no absorbed, precipitated or coagulated amorphous matter" evidently refers to the fully activated material and not to sludge recently exposed to sewage matters. Likewise the assumption that the sludge particles "are composed entirely of active, growing microscopic organisms" implies that the zoogleal masses, as a whole, are endowed with animate energy. As these gelatinous masses have only been studied bacteriologically such a concept of zoogleal activity, although generally accepted, has probably accrued by default.

Parsons (1929) proposed an enzymatic theory coupled with "direct cell action" in explanation for the rapid removal of organic matters in sewage clarification. The protozoa are not mentioned and, as it appears unlikely that colloidal matter is taken up directly by bacteria, enzymes are introduced to account for the breaking down of larger particles. Substances in true solution are then transferred to the living organisms by osmosis. Parson's hypothetical enzyme, like Buswell's, is a lytic agent and not the clotting enzyme of the earlier chemists.

"Direct cell action" is assumed in Parson's theory of clarification as well as in the older theories of biological action. It is difficult, however, to visualize "direct cell action" by bacteria because of the intervening gelatinous matrix in which most of the bacteria are embedded. On the other hand "direct cell action" by the protozoa etc., is an undeniable fact. Thus it is certain that to some extent the coarser particles of raw sewage are attacked and disintegrated by various species

of plankton. Furthermore, the comminuted material may be visibly ingested by these relatively large organisms. By "coarser particles" in this connection are meant particles barely visible to the unaided eye. Suspended matters of greater dimension are, of course, infrequent even in crude sewage and they should be absent from the screened or settled sewage actually applied to filters or mixed with activated sludge.

Opinions as to the importance of protozoa in the clarification of sewage have ranged from the emphatic views of their usefulness expressed by Cramer (1931) to the suggestion by Fairbrother and Renshaw (1922) that the protozoa in activated sludge might be eliminated through the use of certain dyes because of their interference with the process. Butterfield, Purdy and Theriault (1931) have held that the primary function of the protozoa is to prevent the bacterial population from reaching a stalemate, thereby stimulating bacterial growth, with accompanying oxidation. The validity of the more restricted theories of biological action is nevertheless conditioned by the fact that the clarification of sewage, as defined in this paper, cannot very well be credited to the activities of either the bacteria or the protozoa. The action of the protozoa in the disintegration of coarser food particles and in the removal of bacteria from the sludge liquor must be regarded as a continuing process which is certainly not circumscribed by any 30-minute limitation. In the practical absence of quantitative data, the most favorable view of the matter would be to credit the protozoa with the removal of the excess population of bacteria from the body of the liquid. In this sense the protozoa should be credited with a secondary clarifying effect. The disintegration of coarser particles is probably accomplished by the protozoa only after these suspended matters have been agglomerated as a result of the primary clarification of the sewage.

As to the action of enzymes in the disintegration of coarser particles, it is tempting to replace Parson's hypothetical lytic agent by the protozoa although bacterial enzymes are specifically mentioned by him in his important paper. Endo-enzymes may account for the intracellular liquefaction or lysis of ingested food particles. There is no evidence, however, of the existence of exo-cellular enzymes capable of liquefying suspended matter in 30 minutes or thereabouts. From the nature of the sludge floc any enzymes liberated by the bacteria, unless greatly diluted, should be stored in the gelatinous matrix. Lytic action should only be expected after the suspended matters have been collected on the floc. It should also be considered that the definite isolation of enzymes from the sewage or sludge has never been accomplished.

Theriault and McNamee (1930), in experiments already referred to, investigated the possibility that the extraordinary rate of apparent purification or clarification which obtained in their artificial stream was in

reality an oxidation induced by the presence of a relatively high concentration of enzymes in the biological slime (sludge mat) deposited on the stream bed. The hypothetical enzyme visualized in these experiments was therefore an oxidase and not a clotting nor a lytic agent. Clarification by bacteria, or by combinations of bacteria and plankton, had been ruled out by known facts concerning their rates of oxidation. Reasons existed, however, for suspecting the existence of a more rapid oxidation process, loosely described in terms of the then ill-defined "immediate" oxygen demand of sewage and of sludges and tentatively ascribed to enzymatic action. Using the apparatus described by Theriault and Butterfield (1929), complete deoxygenation curves were accordingly obtained from which the respective rates of satisfaction of the "immediate" oxygen demand and of the oxygen demand related to biological activity could be deduced.

TABLE I.—*Oxygen Demand of Sludge*

Period of Incubation	Hours									
	1	2	4	6	11	23.5	29.5	47	74	95
	Oxygen Demand—P.P.M.									
Rapid Process (Y_2)	26	49	86	113	154	185	188	190	190	190
Slow Process (Y_1)	8	16	32	48	87	177	217	325	465	556
Sum [1] (Y)	34	65	118	161	241	362	405	515	655	746
Observed	35	59	120	150	246	356	399	513	663	740

[1] Calculated by the formula,

$$Y = Y_2 + Y_1 = 190.3\ (1\text{–}10^{-0.003271}) + 1088.2\ (1\text{–}10^{-0.05505}).$$

As shown in Table I the disappearance of atmospheric oxygen could very accurately be described in terms of two concurrent reactions, the slower one corresponding to the normal rate of bacterial oxidation and reaching an asymtote in 20 days, and a much faster reaction completed in 20 hours or thereabouts. The results were therefore disappointing in respect to the existence of a major oxidation process which proceeded to 90 per cent completion in 20 minutes. The experimental conditions in these experiments were such that the conclusions as to the rates of oxidation could be checked by striking a very satisfactory balance between the observed decrease in the organic matter content of the sewage and the increased oxygen demand of the sludge mat. It is significant that the rate of bacterial oxidation deduced from these experiments accords satisfactorily with that observed in repeated experiments with

river waters. The rate of satisfaction of the so-called "immediate" demand does not exclude certain types of enzymatic action. The assumption regarding the presence of exo-cellular enzymes in the sludge mat therefore remains a plausible but undemonstrated possibility. The existence of this relatively rapid process of oxidation is undeniable and the same effect has since been repeatedly observed with sludge drawn from an activated sludge plant.

Absorption effects were invoked by Theriault and McNamee (1930) in explanation of the rapid clarification of sewage by activated sludges.

Summary

It is apparent from the foregoing review that definiteness of statement in regard to the various phenomena of sewage treatment implies a recognition of the all-important element of time. With the information at hand regarding underlying rates, it has become possible clearly to separate the basic phenomena of sewage treatment into processes which approach completion, respectively, in days, hours or minutes. On this basis, it is evident that the participation of the biological elements as a major factor in the primary clarification of sewage is definitely ruled out, unless the difficult hypothesis is introduced of direct absorption by bacteria, etc., without accompanying oxidation. If the clarification process is referred to enzymatic action or to the activities of protozoa, it is still necessary to provide a mechanism for the rapid transfer of organic matters to the enzyme substrate or to the protozoan nidus, in either case, the sludge floc.

From various angles, therefore, attention should be focussed on the floc itself as the primary absorbent, apart from embedded bacteria, secreted enzymes or attending protozoa. Such a view of sewage clarification, while plausible enough, raises a difficult question in regard to the nature of the gelatinous matrix which has hitherto been generally regarded as an indissoluble component of the bacteria. New light on this seemingly intractable problem is afforded by the recent announcement (Public Health Reports, Feb. 1, 1935) that the absorbent principle in activated sludge had been definitely identified as a base-exchanging substance, chemically the same as the zeolites of water purification. The data substantiating this proximate analysis of the gelatinous matrix in activated sludge will be presented in the next paper of this series. A discussion of numerous and more or less obvious implications as regards the theory or practice of sewage treatment will be given in succeeding papers.

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PURIFICATION OF SEWAGE BY AERATION*

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Oxidation of sewage with and without activated sludge has been studied in our laboratories, and the results reported (2). It was found that in the absence of activated sludge the reduction of suspended solids and B.O.D. with six hours of aeration was negligible. With longer periods of aeration considerable reductions in suspended solids and B.O.D. took place. These measurements were made on the sewage after aeration, without settling, and therefore did not include the effect of air on purification of sewage after settling. Under certain conditions, as at Indianapolis (1), plain aeration of sewage may be practiced with advantage. Due to overloaded condition of the aeration tanks it was found more economical to aerate all of the sewage, without activated sludge, instead of treating a portion of the sewage by the activated sludge process and subjecting the remainder to plain sedimentation. The removals with plain aeration of all the sewage were 64 per cent more suspended solids and 43 per cent more B.O.D. than the treatment of a part of the sewage by the activated sludge process and the remainder by plain sedimentation. Increased aeration period realized by the failure to return sludge, partially compensated for the absence of the activated sludge. In the light of some of our earlier laboratory results and those reported by others, it was thought advisable to determine under controlled laboratory conditions the actual effect of aeration on the removals of solids and B.O.D., and also to determine whether in the absence of activated sludge the increased concentration of fresh solids in the sewage would prove of any beneficial value. The latter is a point of added interest since theoretically it would establish the necessity for the presence of active floc in the purification process. Practically it would show how variations in the strength of sewage affect the purification brought about by plain aeration.

Method

The method used was to increase the suspended solids of sewage by adding different amounts of freshly settled solids. Suspended solids and B.O.D. were determined on the mixtures before and after settling. The

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TABLE I.—*Effect of Aeration of Sewage (1) and Sewage Plus Settled Solids (2) Mixtures on the Removal of Suspended Solids*

Column	Before Sedimentation P.p.m. 1	After Sedimentation (3) P.p.m. 2	After Aeration (4) & Sedimentation P.p.m. 3	After Aeration Over Corresponding Settled Mixtures Per Ct. 4	After Aeration Over Original Settled Sewage Per Ct. 5	Due to Sed. Per Ct. 6	Due to Sed. & Aer. Per Ct. 7	Due to Aer. Per Ct. 8
1. Sewage Alone	455	300	175	41.7	41.7	34	61	27
2. Sewage & S.S.	965	430	160	63.0	46.7	55	83	28
3. Sewage & S.S.	1415	550	260	53.0	13.3	61	82	21
4. Sewage & S.S.	2180	605	290	52.0	3.3	72	87	15
5. Sewage & S.S.	3400	765	375	51.0	+25.0	77	90	13

(1) Sewage from Freehold, N. J.
(2) Settled solids from Freehold, N. J.
(3) Sedimentation period ½ hour.
(4) Aeration period 6 hours.

mixtures were aerated for 6 hours in a cylinder fitted on the bottom with a fritted glass crucible for air diffusion. After quiescent settling, samples from the supernatant were again taken for suspended solids and B.O.D. determinations.

Results

The results of the first series are given in Table I. The sewage and settled solids obtained from Freehold, N. J., were in a septic condition. The suspended solids of the different mixtures, with and without ½ hour sedimentation, as well as these values after 6 hours aeration and ½ hour sedimentation, are given in the table. Aeration of the sewage resulted in 41.7 per cent removal of the suspended solids in the settled sewage. When the suspended solids content of the aerated mixture was raised to 965 p.p.m. with settled solids, aeration plus sedimentation removed 63.0 per cent more than plain sedimentation. With still higher suspended solids concentrations, the removal due to aeration varied between 51 and 53 per cent. In these calculations the assumption was made that the settled solids added were equivalent to an increased amount of suspended solids in the sewage treated. With such an assumption, results indicated that the removal after aeration was greater than by sedimentation alone. If it be assumed that the sewage to be treated had a definite suspended solids concentration, which was increased by returning various amounts of fresh solids, the net effect of aeration on the removals of suspended solids of the original sewage was somewhat lower; the percentage reductions decreased as the suspended solids concentration of the aerated mixture increased. With the highest suspended solids concentration in the mixture, the effluent after aeration was poorer than with plain sedimentation of the original sewage. The suspended solids reductions due to sedimentation, and due to sedimentation plus aeration, are given in the last three columns of Table I. The difference between the corresponding values represents the reduction due to aeration. The percentage reductions due to sedimentation (column 6) increased with higher original suspended solids concentration—a fact commonly acknowledged. The percentage reductions due to sedimentation and aeration (column 7) were also of an increasing order with increasing suspended solids concentrations. On the contrary, the maximum reductions due to aeration (column 8) were obtained with the two lowest suspended solids concentrations.

The B.O.D. results of the supernatant liquor after sedimentation and the corresponding values after aeration, are given in Table II. The B.O.D. of the supernatant after sedimentation (column 1) increased with increasing concentration of suspended solids in the mixtures. These values were also of increasing order after 6 hours aeration (col-

umn 2) but even the highest B.O.D. after aeration was lower than the B.O.D. of the original settled sewage. The percentage reductions, after

TABLE II.—*Effect of Aeration on Sewage (1) and Sewage Plus Settled Solids (2) Mixtures on The Removal of B.O.D.*

Column	After Sedimentation (3) Ppm. 1	After Aeration (4) & Sedimentation Ppm. 2	After Aeration Over Corresponding Settled Mixtures, Per Ct. 3	After Aeration Over Original Settled Sewage Per Ct. 4
1. Sewage Alone	555	220	60	60
2. Sewage & S.S.	600	230	62	58
3. Sewage & S.S.	620	260	58	53
4. Sewage & S.S.	770	345	55	38
5. Sewage & S.S.	720	415	42	25

(1) Sewage from Freehold, N. J.
(2) Settled solids from Freehold, N. J.
(3) Sedimentation period ½ hour.
(4) Aeration period 6 hours.

aeration of the various mixtures (column 3), calculated on the basis of corresponding B.O.D. values after sedimentation, decreased with increasing suspended solids in the original mixture. The percentage reductions after aeration, when based on the B.O.D. of the original settled sewage, showed an even greater reduction (column 4) with the higher suspended solids in the mixtures.

For the second series of tests fresh weak sewage was obtained from the Princeton, N. J., sewage disposal plant and settled solids from the Red Bank, N. J., plant. The settled solids were fresh and were obtained from heavily chlorinated sewage. The settling period in this and subsequent series was one hour. The results are given in Tables III and IV. With the exception of the mixture with the highest concentration of suspended solids, the suspended solids in the effluent after aeration (Table III, column 3) were consistently lower than the suspended solids in the supernatant after settling alone. The reductions of suspended solids, based on the settled mixtures showed decreasing percentages (Table III, column 4) with higher concentrations of suspended solids. With the exception of only the smallest addition of fresh solids, the effluents of the different mixtures after aeration and sedimentation (Table III, column 5) had higher suspended solids than the original settled sewage. The higsest percentage reduction of suspended solids

TABLE III.—*Effect of Aeration of Sewage (1) and Sewage Plus Settled Solids (2) Mixtures on The Removal of Suspended Solids*

Column	Before Sedimentation P.p.m. 1	After Sedimentation (3) P.p.m. 2	After Aeration (4) & Sedimentation P.p.m. 3	After Aeration Over Corresponding Settled Mixtures Per Ct. 4	After Aeration Over Original Settled Sewage Per Ct. 5	Due to Sed. Per Ct. 6	Due to Sed. & Aer. Per Ct. 7	Due to Aer. Per Ct. 8
1. Sewage Alone	150	100	70	30	30	33	53	20
2. Sewage & S.S.	705	120	85	29	15	83	88	5
3. Sewage & S.S.	1170	160	130	19	+ 30	86	90	4
4. Sewage & S.S.	1855	200	180	10	+ 80	90	90	0
5. Sewage & S.S.	2895	325	265	18	+165	90	91	1
6. Sewage & S.S.	4860	380	705	+86	+605	92	85	−7

(1) Sewage from Princeton, N. J.
(2) Settled solids from Red Bank, N. J.
(3) Sedimentation 1 hour.
(4) Aeration 6 hours.

TABLE IV.—*Effect of Aeration of Sewage (1) and Sewage Plus Settled Solids (2) Mixtures on the Removal of B.O.D.*

Column	Before Sedimentation P.p.m. 1	After Sedimentation (3) P.p.m. 2	After Aeration (4) & Sedimentation P.p.m. 3	After Aeraton Over Corresponding Settled Mixtures Per Ct. 4	After Aeration Over Original Settled Sewage Per Ct. 5	Due to Sed. Per Ct. 6	Due to Sed. & Aer. Per Ct. 7	Due to Aer. Per Ct. 8
1. Sewage Alone	220	225	120	47	47	0	45	45
2. Sewage & S.S.	395	230	110	52	51	42	72	30
3. Sewage & S.S.	510	225	160	29	29	56	70	14
4. Sewage & S.S.	760	255	205	20	9	66	73	7
5. Sewage & S.S.	980	325	270	17	+20	67	72	5
6. Sewage & S.S.	1520	415	390	6	+73	73	74	1

(1) Sewage from Princeton, N. J.
(2) Settled solids from Red Bank, N. J.
(3) Sedimentation 1 hour.
(4) Aeration 6 hours.

TABLE V.—*Effect of Aeration of Sewage and Sewage (1) Plus Settled Solids (2) Mixtures on the Removal of Suspended Solids*

Column	Before Sedimentation P.p.m. 1	After Sedimentation (3) P.p.m. 2	After Aeration (4) & Sedimentation P.p.m. 3	After Aeration Over Corresponding Settled Mixtures Per Ct. 4	After Aeration Over Original Settled Sewage Per Ct. 5	Due to Sed. Per Ct. 6	Due to Sed. & Aer. Per Ct. 7	Due to Aer. Per Ct. 8
1. Sewage Alone	320	210	160	24	24	34	50	16
2. Sewage & S.S.	850	290	265	9	+ 26	66	69	3
3. Sewage & S.S.	1435	415	305	26	+ 45	71	79	8
4. Sewage & S.S.	2365	525	390	26	+ 86	77	83	6
5. Sewage & S.S.	3485	615	565	8	+170	82	84	2
6. Sewage & S.S.	5640	825	965	17	+360	85	83	−2

(1) Sewage from Plainfield.
(2) Settled solids from Plainfield.
(3) Sedimentation 1 hour.
(4) Aeration 6 hours.

TABLE VI.—*Effect of Aeration of Sewage (1) and Sewage Plus Settled Solids (2) Mixtures on the Removal of B.O.D.*

Column	Before Sedimentation P.p.m. 1	After Sedimentation (3) P.p.m. 2	After Aeration (4) & Sedimentation P.p.m. 3	After Aeration Over Corresponding Settled Mixtures Per Ct. 4	After Aeration Over Original Settled Sewage Per Ct. 5	Due to Sed. Per Ct. 6	Due to Sed. & Aer. Per Ct. 7	Due to Aer. Per Ct. 8
1. Sewage Alone	490	310	290	6	6	37	41	4
2. Sewage & S.S.	815	445	470	+ 5	+ 51	45	42	−3
3. Sewage & S.S.	1015	560	505	10	+ 63	45	50	5
4. Sewage & S.S.	1655	680	565	17	+ 82	58	66	8
5. Sewage & S.S.	2510	870	880	+ 1	+180	65	65	0
6. Sewage & S.S.	3450	1060	1240	+17	+750	69	64	−5

(1) Sewage from Plainfield.
(2) Settled solids from Plainfield.
(3) Sedimentation 1 hour.
(4) Aeration 6 hours.

due to aeration was effected in the original sewage. With higher concentrations of suspended solids in the initial mixture, the reductions of suspended solids due to aeration decreased. The B.O.D. values (Table IV) after aeration and settling (column 3) were persistently lower than the corresponding values by plain sedimentation. The addition of fresh solids to the sewage gave effluents, after aeration, which had lower B.O.D. values than the original sewage (with the exception of the two highest concentrations). The percentage reduction of B.O.D. due to sedimentation (column 6) increased with an increase in the original suspended solids concentration. The difference in the percentage reduction obtained by plain sedimentation and aeration plus sedimentation decreased (column 8), with increasing concentrations of suspended solids in the original mixtures.

In the third series the sewage and the settled solids, both in fresh condition, were obtained from the Plainfield, N. J., treatment plant. The results are given in Tables V and VI. Once more the addition of fresh solids to sewage in varying amounts produced effluents after aeration and sedimentation with lower suspended solids content (Table V, column 3) and B.O.D. (Table VI, column 3) than the corresponding mixtures after plain sedimentation. The exception to this was the highest concentration, the effluent of which, after aeration, had a higher B.O.D. and suspended solids content than the same mixture after plain sedimentation alone. However, the various mixtures after aeration gave effluents which in all cases had higher B.O.D.'s and suspended solids than the original settled sewage. The percentage reductions of suspended solids and B.O.D. due to settling (column 6) increased, as in the previous series, with increasing concentrations of solids in the mixtures. The percentage reductions of suspended solids due to aeration (Table V, column 8) decreased with increasing concentrations of solids in the mixture, while the percentage reductions of B.O.D. (Table VI, column 8) were erratic.

Discussion

Consideration of the results of the foregoing experiments leads to the conclusion that when sewage is aerated prior to settling a greater percentage of removal of suspended solids and B.O.D. is obtained than by plain sedimentation. The total suspended solids reductions due to aeration plus sedimentation were 61, 53, and 50 per cent sewages from three sources containing originally 455, 150, and 320 p.p.m., respectively, of suspended solids. The corresponding values obtained by plain sedimentation were 34, 33 and 34 per cent. Therefore, the net effect of aeration was the removal of 27, 20 and 16 per cent of additional suspended solids. The effect of air on the reduction of B.O.D. from sewage was

variable. In the first series aeration resulted in 60 per cent reduction of B.O.D. In the second series no reduction of B.O.D. was obtained by sedimentation, but after aeration and sedimentation 45 per cent reduction was effected. In the final series, sedimentation gave 37 per cent, while aeration plus sedimentation produced 41 per cent reduction of the B.O.D. Thus the overall effect of aeration and sedimentation on B.O.D. reduction varied between 60 and 41 per cent but the net effect of air varied between 60 and 4 per cent reduction. Aeration of sewage will produce, therefore, a greater degree of purification than can be obtained by plain sedimentation. The magnitude of the removals by sedimentation after aeration will depend on several factors. The most important of these is the original suspended solids concentration of the sewage. With a weak sewage, such as in the second series where there was no B.O.D. removal by sedimentation, there was considerable removal after aeration. With concentrated sewage, on the other hand, where the B.O.D. removal due to sedimentation was high, there was little additional removal after aeration. This effect was brought out more clearly when the suspended solids concentrations of the sewages were artificially increased by the addition of settled solids. In such cases the percentage removal due to sedimentation increased so that aeration in the presence of settled solids did not result in appreciably greater removals. Here again the results were influenced by the character of the settled solids and the sewage employed. If the sewage was weak originally, increasing the suspended solids concentration by the addition of settled solids resulted in a greater removal by sedimentation, and consequently in a smaller additional removal after aeration. In the different series there were considerable variations in the removals brought about by aeration. However, with the exception of the highest concentrations of suspended solids, the effluents after aeration had a lower B.O.D. and suspended solids content than effluents from the corresponding concentrations subjected to plain sedimentation. Therefore, aeration of sewage with a strength beyond the normal range is beneficial and will result in greater removals than can be obtained by plain sedimentation. However, the practice of returning settled solids to the sewage for the purpose of increasing the removals is not advisable, since the effluent from such a procedure will be poorer than the effluent from plain sedimentation of the unmodified sewage.

From a theoretical standpoint settled solids cannot be substituted for activated sludge to obtain greater clarification. The floc produced by air has properties not possessed by settled solids. Although by returning fresh solids, removals of suspended solids after aeration and sedimentation may be as high as 70 to 90 per cent and of B.O.D. 50 to

70 per cent depending on the actual amount of settled solids returned, the effluents will still contain high amounts of suspended solids and B.O.D. This is due to the fact that returning settled solids increases the amount of suspended solids and of B.O.D. in the supernatant because of soluble and colloidal materials present in the suspension of settled solids.

It may be well to point out that the results obtained in these studies check the rather high purification reported in a previous paper (3). Solids separated from unaerated sewage yielded a high degree of purification when aerated with sewage.

Nauta (4) reported that the strong Amsterdam sewage, which required a minimum of 14 to 15 hours of aeration in the presence of activated sludge to yield 95 per cent purification, gave 70 per cent purification in 3½ hours. Returning sludge produced in 3½ hours of aeration prevented bulking, which occurred with the longer period of aeration.

It should also be noted that the increased removals obtained by aeration were not due to oxidation, since the reductions of suspended solids and B.O.D. in six hours of aeration, without sedimentation, were negligible. These higher removals were brought about by the flocculating effect of air. It has been shown as a result of studies conducted by the Water Pollution Research Board (5) that bubbling gases such as air, oxygen, hydrogen and nitrogen through sewage caused a flocculation and coagulation and that there were no appreciable difference in six hours in the effects of the four gases used.

Summary

An attempt was made to determine the effect of aeration on the degree of purification of sewage. It was found that with different sewages an aeration period of six hours, followed by sedimentation, resulted in an appreciably greater degree of purification than could be obtained by plain sedimentation. Suspended solids removals of 50 to 60 per cent were obtained with aeration plus sedimentation, as compared with 33 to 34 per cent by plain sedimentation. A greater removal of B.O.D. was also obtained by aeration.

When the suspended solids concentration of the sewage was artificially increased by adding settled solids, the percentage removals by aeration and sedimentation were greater than with the original sewage, although the actual amounts of B.O.D. and suspended solids in the aerated and settled mixtures were higher than in the settled sewage which had not received additions of settled solids. The effluents from such mixtures after aeration and sedimentation had lower B.O.D.'s and suspended solids than the mixtures of corresponding concentration subjected only to plain sedimentation. It may be concluded, therefore,

that aeration of excessively strong sewages will result in an additional purification over that obtained by plain sedimentation, but that settled solids should not be returned to sewage for the purpose of accomplishing a higher degree of purification, because the effluent will be worse than that obtained from the original sewage by plain sedimentation.

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EFFECT OF VARYING THE DAILY CHARGE OF SEWAGE SOLIDS ON THE ACTIVATED SLUDGE PROCESS *

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Continuous aeration of a batch of sewage without the addition of sewage will result in the formation of activated sludge in from 4 to 7 days.[1] With further aeration an over-oxidized sludge will be produced, as evidenced by (1) the continuous loss of suspended matter, (2) the rapid reduction of the B.O.D., and (3) the re-dispersion of the floc. The addition of sewage is therefore necessary (1) to prevent the deterioration of the sludge and (2) to maintain the sludge concentration. The question arises as to how much sewage is necessary to meet these requirements. The weight of raw solids that must be added to meet the loss due to oxidation can be easily determined. However, there is a question whether such an amount of solids is sufficient to maintain the floc in its optimum condition. It is possible that the requirements for these two conditions are different. The problem therefore remains to determine the minimum, optimum and maximum charge of raw solids to activated sludge to maintain it in an active condition.

Present practice in control of the operation of the activated sludge plants is based on the relative volumes of sewage and activated sludge. Since the concentrations of both sewage and activated sludge vary, not only at different plants but also in a given plant at different times, it is to be expected that if the efficiency of the process is determined by the relation of raw solids to activated sludge solids, the volumetric basis of returning sludge may lead to difficulties.

It was, therefore, considered important to determine under laboratory conditions whether (1) the efficiency of the process and the character of the sludge were affected by the relation between the relative amounts of raw solids to activated sludge solids and whether (2) variations of this relationship affect the efficiency and the character of the sludge.

Methods

The activated sludge necessary for the experiments was obtained in two ways: (1) from sewage aerated in the laboratory and (2) from the activated sludge plant in Tenafly. In the first case, 100 liters of concentrated sewage was introduced into the aeration tank (26 by 10 by 30

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inches), and aerated for a period of 2 days, before additions of sewage were begun. The charges were based on the suspended solids of the tank contents, which were low in percentage initially and gradually increased. In order to prevent the suspended solids concentration of the mixture from increasing above certain limits some of the mixture was occasionally withdrawn. The procedure was essentially the same when the experiment was started with activated sludge. In this case the daily charges and withdrawals were started immediately.

Analyses were made to enable us to keep a check on the amount of solids added and withdrawn and to determine the character of the sludge by the B.O.D. and sludge index. The degree of mineralization and nitrification were also determined. The purifying power of the sludge was measured by the removals of B.O.D. and suspended solids from the daily charge of sewage.

Results

In Fig. 1 are presented the cumulative weight of suspended solids

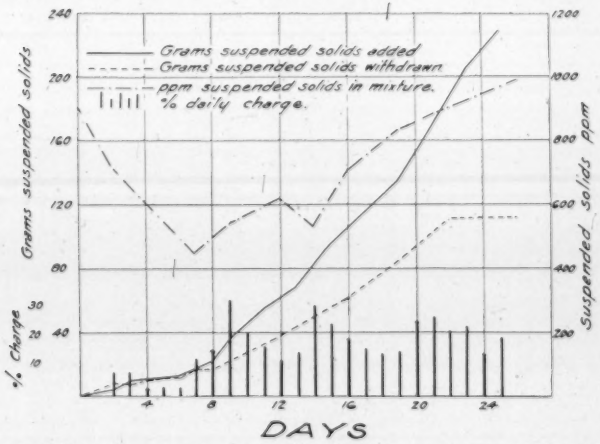

Fig. 1. The aeration of sewage with daily additions and withdrawals and the resultant suspended solids concentration of the mixture.

in the sewage added and in the sludge withdrawn. The daily additions are given, as expressed in per cent of the suspended solids of the tank contents, and also as the resultant suspended solids content of the mix-

ture as determined. During the initial period of 7 days the daily charges were small and were just balanced by the withdrawals. During this period a very rapid decrease of suspended solids in the tank mixture took place. With charges of 5 per cent, the solids concentration could not be maintained if the withdrawals were equal to the additions. The suspended solids concentration of the mixture was steadily increased by increasing the percentage daily charge and by maintaining the withdrawals below the additions. By this procedure the suspended solids concentration of the aeration mixture was brought back to the initial value at the end of 26 days of aeration. The average percentage daily addition for the first week was 3.5 per cent, for the second week 16.5 per cent, for the third week 19.5 per cent and for the final period 19.3 per cent.

A total of 228.9 grams of suspended solids was added in the daily additions, as compared with 87.8 grams in the original sewage. There was left in the tank at the end of the aeration period 95.4 grams of suspended solids after the withdrawal of 111.5 grams during the experiment. Thus, out of 316.7 grams of suspended solids handled, 206.9 grams could be accounted for, resulting in a loss of 109.8 grams. On the basis of the total amount of suspended solids handled this would amount to 34.7 per cent loss for the entire period, or a loss of 1.3 per cent per day. The average daily charge for the entire period amounted to 14.2 per cent, the minimum and maximum daily charges were 2.7 and 30.4 per cent, respectively.

An attempt was made to show how differences in the charging rate affected the sludge characteristics. The results of sludge analyses are given in Fig 2. The sludge index for the solids separated from the initial raw sewage was low. It increased (i.e., the sludge became lighter) with aeration and with increasing concentrations of suspended solids. The maximum value of 95 comes within the range from normal average sludge as given by Mohlman.[2] The nitrate content was low during the first 7 days of aeration. Thereafter it increased rapidly. Two definite decreases took place, each following a maximum charge of the previous day, but the recovery from these set-backs was rapid. The B.O.D.—suspended solids ratio was high initially, decreased to a low value in 5 days and thereafter was not materially affected by increasing daily charges. The values after the fifth day were at or below 0.3. The percentage of ash in the sludge increased rapidly during the first 5 days of aeration, during which time the average daily additions of sewage were low. This corresponded with the period when the suspended solids concentration of the mixture was decreasing rapidly. Between the fifth and ninth days the percentage of ash in the sludge remained at a high level (30 per cent). During this period the charges

were higher than during the previous period. With still higher daily charges the ash content of the sludge decreased to 26 per cent. The turbidity of the supernatant liquid decreased rapidly during the first few days of aeration and continued to decrease at a slower rate with

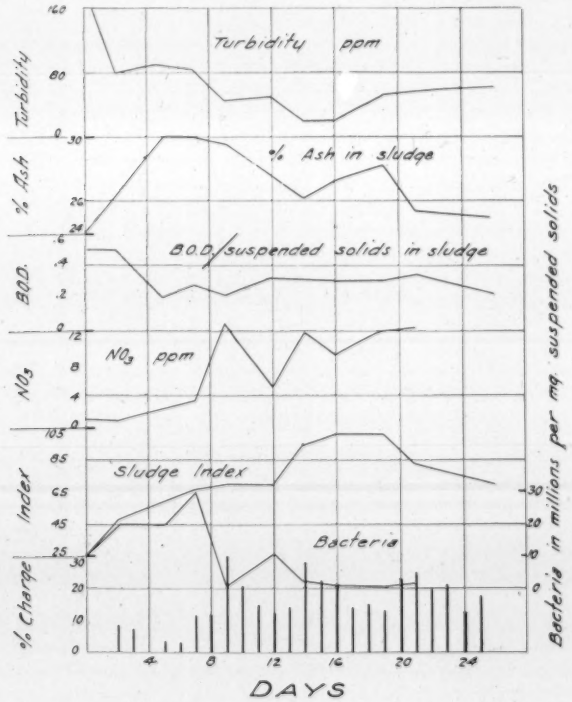

FIG. 2. Some sludge characteristics in the aeration of sewage with daily additions of sewage.

increasing daily additions until a low value of 20 p.p.m. was attained. Thereafter it increased again up to the general level of 50 p.p.m.

In the next experiment, sufficient return sludge from Tenafly was introduced into the tank, which when diluted to 50 liters with water and sewage gave an initial suspended solids concentration of 1910 p.p.m.

Aeration was continued for a period of 6 days with additions of sewage and withdrawals of sludge. The daily procedure was: (1) to withdraw a pre-determined quantity of the mixture, (2) let the mixture settle, (3) draw off the supernatant, (4) add the necessary amount of sewage and make it up to volume with water. The results are given in Table I.

TABLE I.—*The Amount of Solids Added and Withdrawn Daily in the Operation of Activated Sludge Tank*

Days	Suspended In Mixture	Solids In Tank	Suspended Solids Added in Sewage		Suspended Solids Withdrawn
	Ppm.	Gms.	Gms.	Per Ct.	Gms.
0	1910	95.5	2.3	2.4	3.8
1	1880	92.0	2.1	2.3	1.9
2	1800	88	4.2	4.8	1.8
3	1715	84.0	3.0	3.6	1.7
4	1615	79.0	3.0	3.8	1.6
5	1610	79	2.75	3.5	1.6
6	1560	78	—	—	
Total			17.3		12 4
Average				3.4	

The suspended solids decreased rapidly with the average daily charge of 3.4 per cent, in spite of the fact that the total withdrawals were less than the additions. In calculating the amount of suspended solids in the tank each day, the amount withdrawn is not included. The sum of the suspended solids originally present in the tank, plus the total of the daily additions, was 112.8 grams. The suspended solids in the total withdrawals and left in the tank at the end of the experiment was 90.4 grams, giving a loss of 22.4 grams or 20 per cent of the total suspended solids handled. The daily loss of suspended solids was 3.3 per cent. The amount lost was greater than that added in the sewage, therefore the sludge at the end of the experiment should be in a more highly oxidized stage than initially.

That this was the case is brought out in Table II. It will be seen that nitrates accumulated and the ash in the sludge increased considerably over the original. The B.O.D.—suspended solids ratio and oxygen consumed—suspended solids ratios decreased. The turbidity decreased initially but increased thereafter, indicating that material was going back into solution and was being dispersed due to over-oxidation. The sludge index did not vary materially.

The average daily addition of sewage was increased in the next experiment to 7.4 per cent, on the basis of suspended solids, for a period of

13 days. The results are given in Table III and Fig. 3. It will be seen that the suspended solids concentration of the aerating mixture was maintained around 2000 p.p.m. during the experiment by controlling

TABLE II.—*The Effect of Average 3.4 Per Cent Daily Charges of Sewage on the Character and Purification Results of Activated Sludge*

Days	NO_2	NO_3	Ash in Sludge	B.O.D. Susp. Solids	O.C. Susp. Solids	Turbidity	Settling Index
	Ppm.	Ppm.	Per Ct.				
0	.04	.2	27.9	.47	.43	70	105
1	.37	5.6	30.0	.35	.40	30	106
2	.15	8.0	30.5	.35	.34	25	103
3	.15	11.2	32.9	—	—	55	106
4	.12	12.0	34.5	.24	.31	45	112
5	.08	8.0	36.0	.26	.38	40	110
6	.07	7.0	35.0	.19	.32	45	109

the amount wasted. The daily additions varied from a minimum of 4.7 to a maximum of 12 per cent. The weight of solids in the supernatant liquor displaced every day was added to that wasted in the mixture.

TABLE III.—*The Amounts of Solids Added and Withdrawn Daily in the Operation of the Activated Sludge Tank*

Days	Suspended Solids				Withdrawn		
	In Mixture	In Tank	Added in Sewage		In Mixture	In Effluent	Total
	Ppm.	Gms.	Gms.	Per Ct.	Gms.	Gms.	Gms.
0	2060	103	9.5	9.2	—	—	—
1	1960	96	7.7	8.1	1.96	.8	2.76
2	2095	102	12.2	12.0	2.09	.7	2.79
3	2200	108	6.5	6.0	2.20	.4	2.60
4	2200	108	6.9	6.4	2.2	.5	2.70
5	—	—			—	—	—
6	2060	101	10.1	10.0	2.0	.7	2.70
7	2100	103	9.9	9.6	0	0	0
8	2270	113	9.0	8.0	2.3	.35	2.65
9	2295	112	8.2	7.3	2.3	.44	2.74
10	2465	120	8.8	7.3	4.9	.16	5.06
11	2320	111	5.2	4.7	2.3	.28	2.58
12	—	—	—	—	—	—	—
13	2080	100	—	—	4.2	.1	4.30
Total			94.0		26.45	4.43	30.88
Ave.				7.4			

To 103 grams of suspended solids in the activated sludge present initially in the tank, a total of 94 grams of suspended solids was added in the daily additions of sewage. The total of the amount withdrawn and that

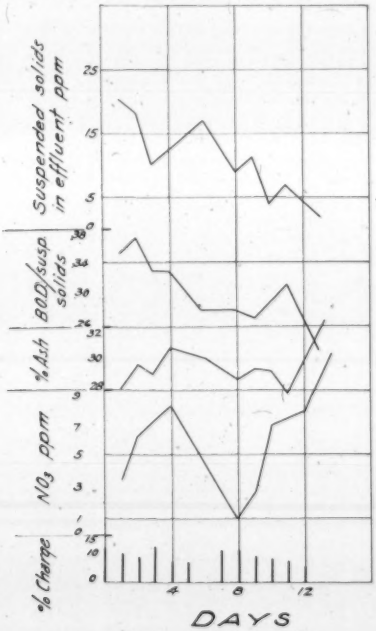

FIG. 3. Some sludge characteristics in the operation of the aeration unit with daily charges of sewage averaging 7.4 per cent (on the basis of suspended solids of the tank content).

left in the tank was 130.9 grams, giving a loss of 56.1 grams. The loss during the 13 day period was 28.5 per cent of the total amount of suspended solids handled or 2.2 per cent per day. The results as given in Fig. 3 show that neither the character of the sludge nor the purification suffered as a result of the additions. Nitrates were above 3 p.p.m. with one exception. The ash content of the sludge fluctuated between 28 and 30 per cent. The B.O.D.—suspended solids ratio decreased from

0.36 to 0.24 during the course of aeration. The suspended solids of the supernatant decreased from 20 to 2 p.p.m

In the next experiment the daily charges were increased still further (Table IV). The daily suspended solids addition represented an average

TABLE IV.—*The Amount of Solids Added and Withdrawn Daily in the Operation of the Activated Sludge Tank*

Days	Suspended Solids				Withdrawn		
	In Mixture	In Tank	Added in Sewage		In Mixture	In Effluent	Total
	Ppm.	Gms.	Gms.	Per Ct.	Gms.	Gms.	Gms.
0	2082	100	22.4	22.4	—	—	—
1	2425	116	16.7	14.4	4.85	.65	5.50
2	2535	119	20.9	17.5	7.60	.75	8.35
3	2740	126	19.4	15.4	10.65	.40	11.35
4	2915	128	11.8	9.2	77.5	.50	18.0
5	2765	116	18.0	15.5	22.1	0	22.1
7	2455	88	18.8	21.4	34.4	.20	34.6
9	2065	101	16.7	16.5	2.06	.16	2.22
10	2305	101	12.1	12.0	13.8	.88	14.68
11	2115	89	16.9	19.0	16.9	.32	17.22
12	2055	90	27.0	30.0	12.3	1.2	13.50
13	2085	92	20.1	21.8	12.5	1.08	13.58
14	2035	89	29.0	32.6	12.2	1.12	13.32
15	2415	120.7	—	—	—	—	—
Total			249.8				
Ave.				17.7	167.1	7.26	174.42

of 17.7 per cent of the suspended solids in the tank. The minimum daily addition was 9.2 per cent and the maximum 32.6 per cent. The suspended solids concentration of the mixture was maintained at approximately 2000 p.p.m. by increasing or decreasing the withdrawals. To the initial 100 grams of activated sludge a total of 249.8 grams of sewage suspended solids were added. Of this only 295.1 grams were recovered in the wasted mixture and supernatant liquor plus what was left in the tank at the end representing a loss of 54.7 grams of suspended solids. Of the total amount of solids handled 15.6 per cent was lost during the entire period or 1.0 per cent per day.

The effect of these additions on the character of the sludge and purification is represented in Fig. 4. The nitrates dropped initially and later increased gradually towards the end of the period. The ash content of the sludge decreased from 32 to 24 per cent. The B.O.D.—suspended solids ratio increased from 0.24 to 0.42 in 5 days, dropped

down to 0.24 and then increased gradually to 0.32 at the end of the period. The suspended solids in the effluent decreased from 15 to less

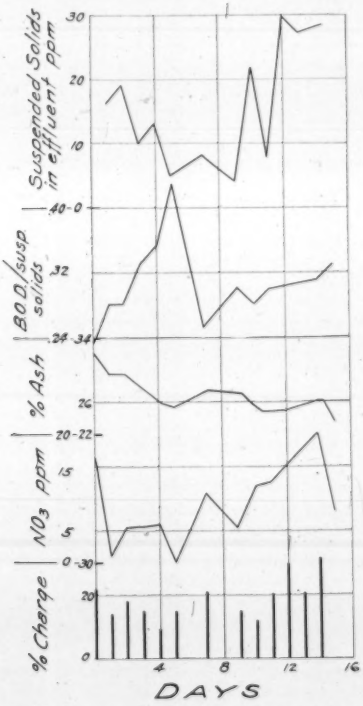

FIG. 4. Some sludge characteristics in the operation of the aeration unit with daily charge of sewage averaging 17.7 per cent (on the basis of suspended solids of the tank content).

than 5 p.p.m. in 5 days and then increased to 30 p.p.m. toward the end of the experiment.

DISCUSSION

The results presented above, obtained in the operation of an experimental activated sludge tank, with daily additions and withdrawals, have confirmed the conclusions derived from previous studies without

daily additions, namely that aeration of activated sludge and sewage results in a loss of suspended solids due to oxidation. Some of the results of the different runs are brought together in Table V.

TABLE V

Run	Daily Charge	Susp. Solids Lost per Day	Ash of Sludge	B.O.D. Susp. Solids	NO_3
	Per Ct.	Per Ct.	Per Ct.		Ppm.
1	14.2	1.3	25–28	.22–.34	4–12
2	3.4	3.3	33–36	.19–.26	7–12
3	7.4	2.2	28–30	.24–.31	1–16
4	17.7	1.0	24–27	.25–.33	10–20

When the daily charge on the basis of suspended solids was 3.4 per cent, the loss per day due to oxidation was 3.3 per cent of the total weight of suspended solids handled. When the charge was increased to 7.4 per cent the loss amounted to 2.2 per cent and with 17.7 per cent addition the loss per day decreased to 1 per cent. These results indicate that the rate of oxidation of the solids was adversely affected by a high rate of charging. It follows that the weight of dry solids wasted will be less than the weight of dry solids received, to the extent of the material that has been oxidized. If the daily charges are less than the solids oxidized, it is obvious that no sludge can be wasted and the sludge concentration in the aeration unit cannot be maintained, because of the oxidation of the sludge itself. This condition will be reflected in an increase of ash in the sludge, a decrease in the B.O.D.—suspended solids ratio and a deterioration of the effluent. With an average daily charge of 3.4 per cent the ash in the sludge established itself around 33 to 36 per cent. When the charge was increased to 7.4 per cent the ash was between 28 and 30 per cent and with a charge of 17.7 per cent the ash content was between 27 and 24 per cent during the period of equilibrium. Similarly the B.O.D.—suspended solids ratio, with 3.4 per cent charge, was between 0.19 and 0.26 toward the end of the period. With 7.4 per cent charge the variation toward the end of the period was between 0.31 and 0.24 and with 17.7 per cent it increased from 0.25 to 0.33 in the latter part of the experiment. Nitrate accumulation seemed to increase directly with the rate of charging. With 3.4 per cent addition nitrates varied from 7 to 12 p.p.m., with 7.7 per cent charge they increased from 1 to 16 p.p.m., and with 17.7 per cent they varied between 10 and 20 p.p.m. It seems plausible that when oxygen is not the limiting factor nitrification will be dependent on the amount of ammonia available, which increases with the higher rates of charging.

The effect of rate of charging on the purification is of interest. With a daily charge of 3.4 per cent the effluent, as measured by the turbidity, deteriorated toward the end of the charging period. This indicates an over-oxidized sludge, resulting in the breaking up of the sludge floc. The humic acids produced by oxidation of the sludge tended to increase the color. With 7.7 per cent daily additions the suspended solids in the effluent progressively decreased toward the end of the experiment. With 17.7 per cent charge the suspended solids in the effluent progressively increased.

In this discussion the changes taking place during the early period have not been emphasized, because they have been attributed to the characteristics of the initial sludge. As the experiments progressed this sludge was gradually replaced by the sludge produced under the conditions of the experiment and the characteristics of this sludge should be considered. It may be that the period of charging in some of the experiments was not long enough to efface completely the effect of the original sludge. With 3.4 per cent charges the total amount of suspended solids added was 17.3 grams and the initial sludge content was 95.5 grams. In spite of this, however, there were definite indications that the sludge was getting over-oxidized. With a daily 7.4 per cent charge, 94 grams of sewage solids were added to the initial 103 grams of sludge. With 17.7 per cent charge, 250 grams were added to an initial 100 grams of sludge.

The results seem to indicate that an average daily charge of 17.7 per cent was excessive and that a charge of 3.4 per cent was insufficient. The optimum should be somewhere between these two rates.

Summary

An aeration tank was operated under laboratory conditions with daily charges of sewage and withdrawals of the mixture of sewage and sludge to maintain a definite concentration of suspended solids. The daily charges were based on the relation between the suspended solids content of the sludge in the tank and the suspended solids of the sewage added. The daily charges varied from an average of 3.4 to 17.7 per cent. The balance between weight of solids added and recovered showed a loss in every case. The percentage of solids lost per day was greatest with the lowest daily charge and lowest with the greatest daily charge. If the daily additions are not sufficient to compensate for this loss the concentrations of solids in the aeration tank cannot be maintained. The character of the sludge and the degree of purification are affected by the rate of charging. If the charge is too small an over-oxidized sludge will be produced, as indicated by the high ash content of the sludge, the low B.O.D.—suspended solids ratio, and the higher

turbidity of the effluent. The latter phenomenon is due to the breaking up of the floc and the presence of brownish colored material of humic acid nature. When the daily charges are too high the sludge has a low per cent ash, a high B.O.D.—suspended solids ratio and the suspended solids in the effluent are high. Nitrification is not adversely affected by high rates of charging, but it is increased because of larger amounts of ammonia in the influent. A daily charge of 3.4 per cent seems too low while a charge of 17.7 per cent seems too high, according to the above standards.

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EFFECT OF COMMINUTION OF SEWAGE SOLIDS UPON THEIR SUBSEQUENT DIGESTION

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The solids settling from sewage by plain sedimentation usually vary widely in composition and size. When they are subjected to anaërobic digestion it would seem that their size as well as their composition should influence their course of digestion, since the biochemical changes involved in this process presumably take place very largely on the surface of the solids. A report on the results obtained from the batch digestion in the laboratory of normal and comminuted sewage solids may therefore be of interest. It may be well to warn the reader, however, that the findings are by no means striking.

Experimental Procedure

A catch of solids settling from a more or less domestic sewage was obtained by hanging buckets in the flowing-through chamber of Imhoff tanks at Fitchburg, Mass. The collections included the characteristic miscellany of paper, fecal matter, vegetable refuse and pipe growths. A portion of this fresh sludge was comminuted by passing it re-

Table I.—*Composition of Sludge Mixtures.*

	Fresh Solids	Digested Solids	Mixture*	Ratio of Fresh to Digested Solids
Weight of Sample—Grams	2250	750	3000	3.0 : 1
Total Solids by Weight—Per Cent	4.46	8.04	5.35	0.6 : 1
Grams*	100.2	60.2	160.4	1.7 : 1
Volatile Solids—Per Cent	75.1	48.5	65.1	1.6 : 1
Grams*	75.2	29.2	104.4	2.6 : 1
Fuel Value of Total Solids—				
Kilogram Calories per Gram	4.85	3.07	4.18	1.6 : 1
B.T.U. per Pound	8720	5520	7630	1.6 : 1
Kilogram Calories*	486.0	184.8	670.8	2.6 : 1
B.T.U.*	1930	733	2663	2.6 : 1

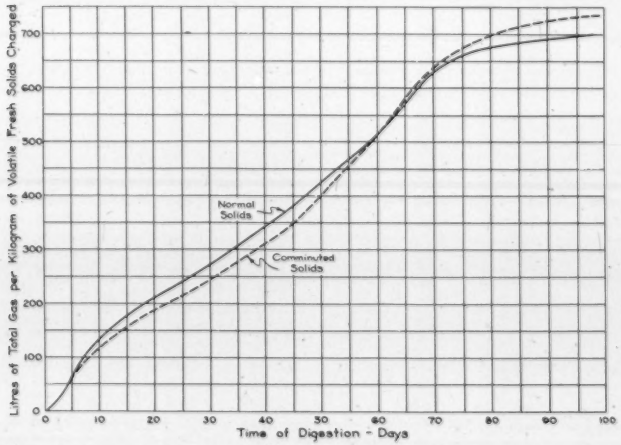
* Computed values.

peatedly through a powerful gear pump until the mass had the consisteney of thick soup. Digested sludge for seeding purposes was obtained from the same source. It was sufficiently finely divided not to require comminution. Equal weights of normal and comminuted solids were mixed with one-third their weight of digested sludge and separately digested at 25° C. (77° F.). The composition of both mixtures was identical and is summarized in Table I.

The mixtures were allowed to digest until gas production became negligible, the volume and composition of the gas being noted during the course of digestion. The equipment employed has been previously described.[1] The digested sludges were analyzed and the various parameters of digestion computed.

Experimental Results

The course of digestion as measured by gas production is shown in the accompanying diagram. For purposes of generalization the gas



Cumulative gas production during the digestion, at 25° C. (77° F.), of normal and comminuted sewage solids.

volumes are expressed in liters per kilogram of volatile fresh solids.

The parameters of digestion are summarized in Table II together with the percentage differences noted for the parallel experiments.

It appears from the diagram and from Table II that comminution had little effect on the general course of digestion, the quantity of volatile matter destroyed and the ultimate volatile content of the digested sludge. At the beginning of digestion, gasification of comminuted solids lagged behind that of the normal sludge. Towards the end of the test, however, the comminuted solids overtook the normal solids in gas

TABLE II.—*Effect of Comminution of Sewage Solids on Parameters of Digestion.*

Parameter	Normal Sludge	Comminuted Sludge	Difference Relative to Normal Sludge %
Volatile Matter Reduction			
Volatile Matter Destroyed—Grams	53.5	53.9	+0.75†
Volatile Matter in Final Sludge—Per Cent	51.0	50.1	−1.8
Ratio of Volatile Matter Destroyed to Volatile Matter in Fresh Solids	0.71	0.72	+1.4
**Gas Production*			
Volume of Total Gas (0° C., 760 mm)—Liters	52.7	55.3	+4.7
Weight of CO_2—Grams	33.4	36.1	+8.1
Weight of CH_4—Grams	25.4	26.3	+3.5
Total Weight of Gas—Grams	58.8	62.4	+6.5
Pounds of Gas per Pound of Volatile Matter Destroyed	1.10	1.16	+5.5
Cubic Feet of Gas (20° C., 760 mm.) per Pound of Volatile Matter Destroyed	16.95	17.65	+4.1
CubicFeet of Gas per Pound of Fresh Volatile Matter Charged	11.2	11.8	+5.36
Cubic Feet of Gas per Pound of Fresh Solids Charged	8.4	8.86	+5.36
Time for Digestion			
Time for 90% of Total Gas Production from Normal Solids—Days	69.7	68.8	−1.3
Fuel Values			
Fuel Value Lost by Solids during Digestion—Kg. Cal.	261.8	272.3	+4.0
Fuel Value of Methane Produced—Kg. Cal.	335.4	347.0	+3.5
Excess of Fuel Value of Methane—Kg. Cal.	73.6	74.7	+1.5
Excess of Fuel Value of Methane—B.T.U. per Pound of Volatile Matter Digested	2480	2500	+0.8
Percentage of Fuel Value of Fresh Solids Recovered as Methane	69.0	71.4	+3.5

* All gas production values represent actual "collectable" gas. They are not corrected for CO_2 retained in the sludge liquor.

† A positive sign denotes an increase in the parameter for comminuted solids, a negative sign conversely a decrease.

production. The final difference in gas volume produced was in the vicinity of 5 per cent. The difference is accounted for largely by an increased evolution of carbon dioxide from the comminuted solids. This relative excess in gas production by the comminuted solids is maintained when gasification is expressed in terms of the volatile matter destroyed rather than the charge of fresh volatile solids or of total fresh solids.

The pounds of collectable gas per pound of volatile solids destroyed (1.10 for normal solids and 1.16 for comminuted solids) compare with the values (1.15 to 1.25) reported for plant and laboratory work by Buswell and others.[2, 3, 4] Expressed as cubic feet of gas per pound of volatile matter added, the results are near the maximum of 11.5 given by Rudolfs.[5] As in the experiments performed by Buswell and Pearson,[4] loss in ash was observed. This loss is due apparently to sampling errors. In the authors' experiments the entire mixture was small enough to be thoroughly agitated during sampling. Even so, however, a quantity of heavy sandy material settled out so rapidly that it probably escaped from the sample taken for analysis. If, in accordance with the method suggested by Buswell and Pearson,[4] we correct for this loss but assume that the material contained about 70 per cent of ash, the values for volatile solids destroyed are reduced from 53.5 and 53.9 grams to 50.5 and 51.5 grams respectively. The parameter "pounds of collectable gas per pound of volatile matter destroyed" is then raised from 1.10 and 1.16 to 1.17 and 1.21 respectively. These values are in still better agreement with those which have been referred to previously. The remaining parameters that are based on volatile matter destruction are increased correspondingly.

An attempt has been made, as in earlier work by the authors,[6] to establish a heat balance for sludge digestion. As shown in Table II, the fuel value of the methane produced during digestion was substantially higher than the difference in the fuel values of the charge and residue of solids. The theoretical significance of this result is a controversial matter[6, 7] which the authors do not desire to discuss before definitive observations are available. The question of fuel values of sludges which is assuming such great importance in modern methods of sludge treatment and disposal as well as the question of "self-heating" of digesting sludge requires further study by many workers. It would appear that the answer must be predicated upon a better understanding of the losses incurred in drying sewage solids. Present standard methods of analysis do not permit obtaining reliable information of the heat quantities that are available in the gas and digested solids in terms of the fuel value of the charge of the solids received by a digestion tank. In the study here reported, the excess of the fuel value of the methane produced over the fuel value lost by the solids during digestion equalled 28.1 and

27.4 per cent of the fuel value lost, or 12.9 and 13.1 per cent of the fuel value of the mixed charge at the beginning of the experiment.

Summary and Conclusions

The difference between the parameters of digestion for normal and for comminuted sewage solids is small but definitely favorable to comminution in all cases. The results reported do not hold the promise, however, that much is to be gained in practice by comminution of the sludge charge of digestion tanks when the solids are permitted to remain at rest, as was the case in the tests here reported. Under these circumstances, the compacting of the sludge settling to the bottom of the tank may be expected to affect much of the theoretically possible benefit to be derived from comminution of the solids. Hence the thought is advanced that agitation of the sludge during digestion may utilize the surfaces created by comminution more effectively and may yield more significant differences than those so far observed.

While the practical significance of the comminution of solids may be small, the authors have been convinced by these and other tests that the process is of decided value in experimental work when much may depend upon the uniformity of the samples employed. Comminution of solids subjected to digestion has become standard practice in their laboratory.

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HIGH AND LOW TEMPERATURE DIGESTION EXPERIMENTS. I. OPERATION AND SEEDING*

By Willem Rudolfs and Henry J. Miles

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We have conducted a series of experiments in a pilot plant at the Freehold, N. J., Sewage Treatment Plant to (1) compare digestion of sewage sludge at mesophilic (80°–85° F.) and thermophilic (120°–125° F.) ranges; (2) obtain information on operation difficulties; (3) determine the shortest possible practical schedule of digestion; (4) determine the effect of mixing and stirring; (5) obtain data on tank insulation; (6) determine the feasibility of combining mesophilic and thermophilic (consecutive) digestion; and (7) find under carefully controlled conditions the quantities of gas, characteristics of sludge produced, nature of supernatant liquid, scum formation, odors, etc. In addition, laboratory experiments were made to determine the relative effects of a number of trade wastes on high and low temperature digestion.

The results presented in this paper deal principally with production of proper seed material, operation difficulties, movement of sludge in the tank, drainability of sludge and some data on gas production with different digestion schedules during the first 10 months of operation.

With the cooperation of the Borough of Freehold, two tanks were built, each with an effective digestion capacity of 140 gallons, estimated to be able to take care of the minimum sludge production of 125 persons. The two identical tanks received portions of the same batches of fresh solids pumped from the clarifier. One tank was kept at a temperature between 80°–85° F., the other between 120°–125° F. They were operated simultaneously, with the difference in temperature as the only variable.

Description of Tanks

Each tank used was made of three sections of vitrified clay sewer pipe 24 inches in diameter, set in concrete bases. The tanks were equipped with sheet metal covers, containing gas domes and hand holes, valves, recording thermometers, hot water heating systems, and after the working-in period with stirring devices. The high temperature tank was also equipped with a scum breaker. A diagrammatic sketch of the tanks and apparatus is shown in Figure 1. The covers were made gas-

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tight by keeping the level of the liquid in the tanks slightly higher than the edges of the cover. The temperature of the tank contents was regulated by varying the amount of gas supplied to the water heater. The gas was collected from the Freehold sludge digestion tank. The stirring

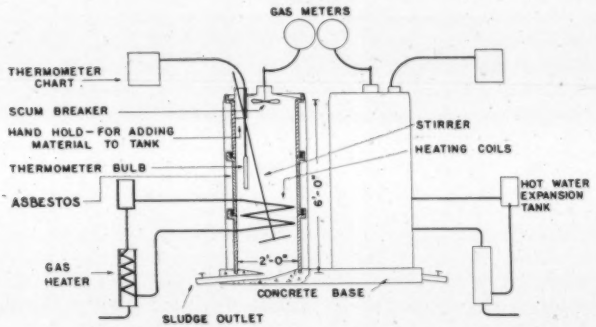


FIG. 1.—Diagrammatic sketch of experimental digestion tanks showing heating systems, thermometers, stirrers, scum breakers, gas collection system, etc.

apparatus consisted of circular metal disks fastened to the end of a pole, which, when raised and lowered several times, thoroughly mixed the contents of the tank. The scum breaker in the high temperature tank was an old electric fan attached to one end of a vertical shaft and was operated by turning a handle on the other end. Both tanks were insulated with a two-inch coating of asbestos, painted with a waterproofing compound. The insulation work was done and donated by the Johns-Manville Corporation. Loss of gas through the hand holes was minimized by having the side walls extend about a foot below the cover. Under these conditions the loss of gas was extremely small.

PROCEDURE

At the start of the experiments both tanks were filled with digesting sludge taken from the sludge digestion tank of the Freehold treatment plant. An analysis of the sludge made at that time showed that it contained 3.3 per cent solids with 41 per cent ash. Special care was taken to have identical sludge in both tanks. The tank contents were then heated to the required temperatures and after one week, daily additions of fresh solids were begun at a rate of 3.5 gallons per day. On a displacement basis this would correspond to a digestion schedule of 40 days.

Before each addition of fresh solids, sufficient supernatant liquid was pumped from the top to make room for the new material. The tank contents were then stirred thoroughly and allowed to remain to settle and concentrate until the next day. All material removed and added was measured by means of pails and samples were taken from each pail. The samples were composited and analyzed at weekly intervals for solids and ash. The fresh solids of the daily additions were obtained from the sumps of the primary settling tanks. Samples were also collected at different depths in the tanks at weekly intervals and analyzed. Gas was measured by means of gas meters connected with each tank and the daily volume produced was recorded.

Operation Difficulties

The above schedule was followed for 39 days. The low temperature tank produced considerable gas, but the high temperature tank very little or none. The pH of the contents of the former tank increased continuously, while that of the latter decreased. The supernatant liquid in the mesophilic tank contained more total solids than the supernatant in the thermophilic and was light straw in color after settling, whereas that from the high temperature tank remained black after the solids had settled. The ash contents of the liquid also differed considerably, that of the former being 38 per cent and of the latter 26 per cent on a dry basis.

The absence of gasification in the thermophilic tank caused considerable stratification. Occasional stirring helped very little in preventing stratification. It was decided to let the tanks rest in an attempt to produce proper thermophilic seed sludge. A two weeks rest was insufficient to correct the existing conditions in the high temperature tank. Excessive scum formation was still present. The solids concentration had become so great that an insulating layer was formed around the heating coils, preventing uniform temperature conditions throughout the tank. Half of the contents of each tank were replaced with fresh sewage. A few days later daily additions of fresh solids at the rate of 3.5 gallons per day were resumed.

During the next month (January, 1934) the extremely cold weather and the exposure of the tanks made it impossible to maintain the desired temperature in the high temperature tank. The two-inch asbestos insulation proved to be insufficient to prevent great heat losses from the high temperature tank, but was enough to allow the maintenance of the desired temperature in the mesophilic tank. The result was that during this period no satisfactory thermophilic seed sludge was produced. In February the required temperatures could be maintained in both

tanks. In spite of the fact that no proper thermophilic seed sludge was available the daily additions were gradually increased in both tanks to find out if (1) proper seeding material would be formed under these conditions in the high temperature tank, (2) improperly seeded thermophilic sludge would digest as much fresh solids as properly seeded mesophilic sludge, (3) what differences in behavior would be most apparent under these conditions.

The gradual increase in daily additions was continued until early August, when the digestion tanks were operating on a displacement schedule of 10 days. At that time a new series of experiments was inaugurated, the results of which will be reported later.

The heavy scum which continually formed on the surface of the high temperature tank had made it necessary to install a scum breaker and a stirring device earlier in the year. In order that the operation results of both tanks might be comparable similar devices were placed in the mesophilic tank. The tanks were stirred on an average once an hour during the afternoon and early morning, the tanks being allowed to remain quiescent during the night and two hours before drawing supernatant liquid at noon. Sludge was drawn from the bottom of the tanks whenever the supernatant fluid became too heavy.

Near the end of the experiment, when the digestion schedule had been gradually reduced to 10 days, the mesophilic tank exhibited excessive foaming. The tank was allowed to rest for one week and daily additions were then resumed on a 17.5 day displacement schedule. The same results were obtained with the sludge in the high temperature tank, but it is very improbable that real thermophilic digestion with proper seed material was ever established during this period of experimentation because shorter digestion schedules were obtained later in the same tank.

Results

The experimental period from October, 1932, to August, 1933, has been divided into two periods, namely from October to March and from April to August. During the winter months with extreme cold weather, which sometimes made it impossible to heat the tanks properly, operation difficulties in the form of frozen gas and liquid piping, and scum formation in the tanks were perhaps the most outstanding results obtained.

The average solids and ash contents and the amounts of volatile matter present in the tanks at the beginning and end of the experiments were as follows:

	Low Temperature		High Temperature	
	Begin	End	Begin	End
Solids, Per Cent	3.74	3.16	3.10	3.09
Ash, Per Cent	40.8	38.0	42.2	23.7
Dry Volatile Matter, Lb.	25.9	23.0	19.8	24.8

A condensed summary of the data for the first period is given in Table I. With a 40-day digestion schedule the material in the low temperature tank produced nearly a normal amount of gas per pound of volatile matter added, whereas the material in the high temperature tank

TABLE I.—*Condensed Results During First Period*

	Low Temperature		High Temperature	
	Oct.–Jan.	Feb.–Mar.	Oct.–Dec.	Feb.–Mar.
Days Run	112	42	62	35
Digestion Schedule, Days	40	31	40	31
Fresh Solids Added, Avg. Per Ct. Solids	4.32	3.23	4.78	3.54
Ash of Fresh Solids, Per Ct.	19.6	19.5	20.0	19.4
Volatile Matter Added, Lb.	85.11	39.07	48.26	36.91
Gas, Cu. Ft.	751.3	418.8	21.1	361.8
Gas per lb. Vol. Matter Added, Cu. Ft.	8.83	10.72	0.47	9.80

produced very little gas. During the second part of this period, when a digestion schedule of 31 days was maintained and difficulties with scum formation had been overcome with the help of stirring apparatus, the amount of gas produced in the high temperature tank was about normal, whereas that produced by the low temperature tank was somewhat high.

The results obtained during the second period from the low temperature tank are graphically shown in Figure 2 and of the high temperature tank in Figure 3. It will be noted that the digestion period was gradually reduced from 31 to 10 days. After the latter schedule had been maintained for 11 days, excessive foaming occurred which made it necessary to stop adding fresh solids. After resting for one week, daily additions were resumed on a 17.5 day schedule. During the second period the sludge concentration and the percentage of ash gradually decreased in the low temperature tank, while the solids concentra-

tions of the supernatant liquor gradually increased. The ash content of the material in the high temperature tank decreased gradually, while the sludge concentration remained practically constant until a period of

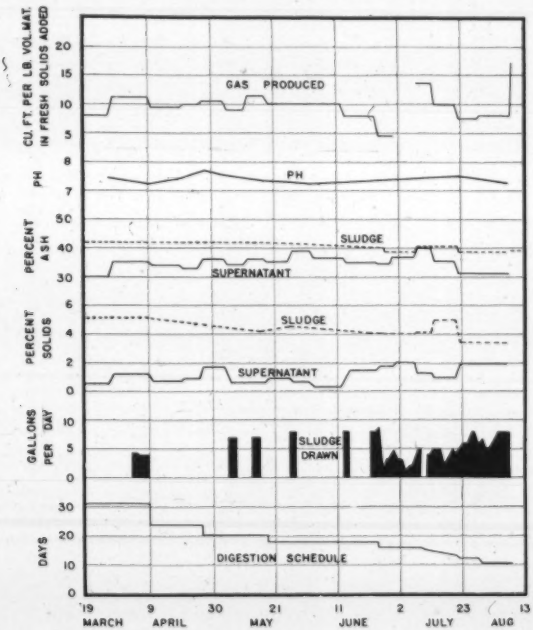


FIG. 2.—Operation results of low temperature tank.

heavy withdrawals was begun. Apparently overdrawing of digested sludge from the high temperature tank was easier than from the low temperature tank, leaving insufficient seed to continue proper operation. The overdrawing was probably due to the great rate of liquefaction of the material in the thermophilic tank.

The distribution and movement of solids differed materially in the two tanks. The condition during the first and second period is graphically shown in Figure 4, while the ash content of the solids at various levels is shown in Figure 5. Shortly after operation had been started the solids in the low temperature tank were uniformly distributed

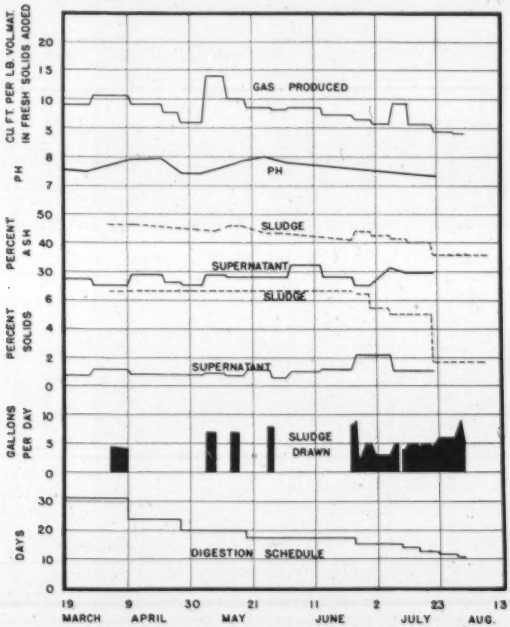


FIG. 3.—Operation results of high temperature tank.

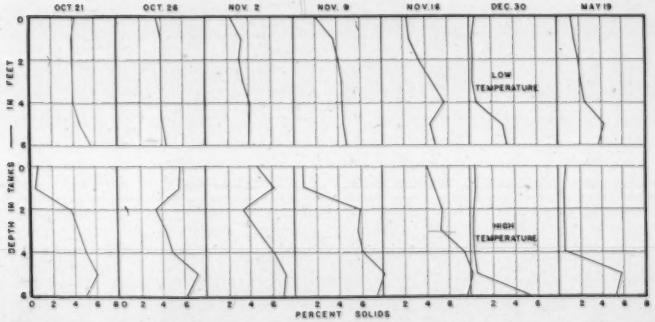


FIG. 4.—Distribution of solids throughout the tanks.

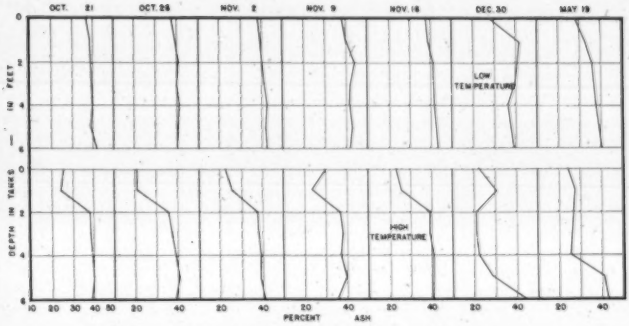


FIG. 5.—Ash content of solids throughout the tanks.

throughout the volume of sludge, while the separation of water in the high temperature tank was much greater (see Figure 4, Oct. 21). During the next week more and more solids began to float in the high temperature tank, while this condition was reversed again after about one month's operation. This manner of sludge movement continued until after about 3½ months of operation stirring devices were installed. The results for ash content show perhaps even better the greater variation in the high temperature tank.

The results obtained during the second period have been averaged and summarized in Table II. The average concentration of the sludge

TABLE II.—*Condensed Results During Second Period*

	Low Temperature Apr.–Aug.	High Temperature Apr.–Aug.
Days Run	122	122
Digestion Schedule, Days	10 to 23.3	10 to 23.3
Av. Fresh Solids Added, Per Ct. Solids	3.88	3.95
Av. Ash of Fresh Solids, Per Ct.	21.4	21.1
Av. Solids in Supernatant, Per Ct.	1.04	1.08
Av. Ash in Supernatant, Per Ct.	35.2	28.4
Av. Sludge Drawn, Per Ct. Solids	4.98	5.92
Av. Ash of Sludge Drawn, Per Ct.	40.6	42.5
Volatile Matter Added, Lb.	265.3	244.3
Volatile Matter Destroyed, Lb.	175.8	170.5
Av. Volatile Matter Destroyed, Per Ct.	66.3	69.7
Gas, Cu. Ft.	1948.5	2037.1
Av. Gas per Lb. Vol. Matter Added, Cu. Ft.	7.35	8.82
Av. Gas per Lb. Vol. Matter Destroyed, Cu. Ft.	11.68	11.78
Weight of Gas, Lb	104.16	85.67
Av. Gas per Lb. Vol. Matter Destroyed, Lb.	0.592	0.502

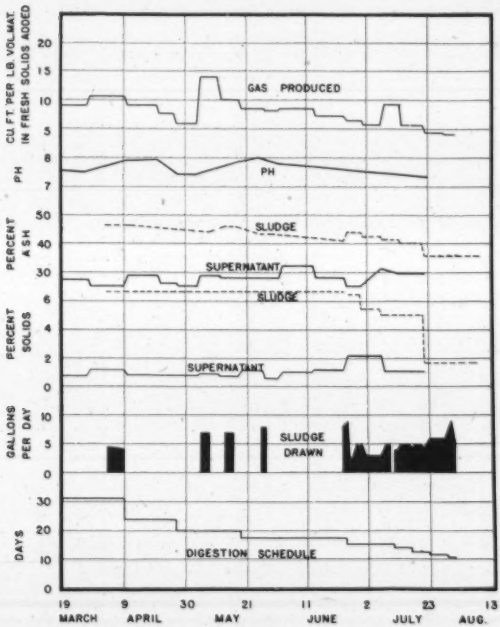
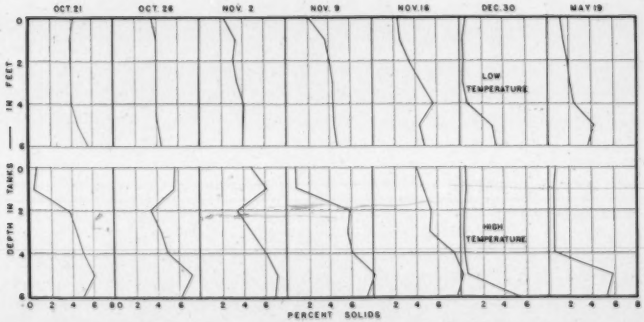


FIG. 3.—Operation results of high temperature tank.



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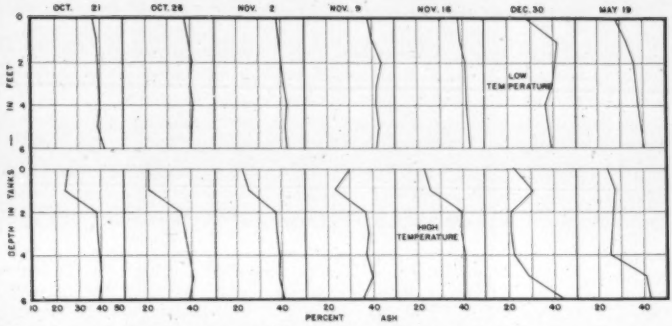

FIG. 5.—Ash content of solids throughout the tanks.

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Gas, Cu. Ft.	1948.5	2037.1
Av. Gas per Lb. Vol. Matter Added, Cu. Ft.	7.35	8.82
Av. Gas per Lb. Vol. Matter Destroyed, Cu. Ft.	11.68	11.78
Weight of Gas, Lb	104.16	85.67
Av. Gas per Lb. Vol. Matter Destroyed, Lb.	0.592	0.502

drawn was about 20 per cent greater from the high temperature tank than from the low, and gas production per pound of volatile matter added was also 20 per cent greater. With the reduction in digestion time, gas production per pound of volatile matter destroyed decreased, while the apparent volatile matter destruction on a percentage basis remained fairly constant. In order to demonstrate the digestion changes from gasification to liquefaction, results obtained during the second period for the different digestoin times were calculated and are given in Table III. It will be seen that gas production, in pounds per

TABLE III.—*Gas Production With Varying Digestion Periods*

Run	Digestion Period	Low Temperature			High Temperature		
		Vol. Mat. Destroyed	Gas. Prod. per Lb. V. M. Destr.	Apparent Vol. Mat. Destroyed	Vol. Mat. Destroyed	Gas. Prod. per Lb. V. M. Destr.	Apparent Vol. Mat. Destroyed
Days	Days	Lb.	Lb.	Per Ct.	Lb.	Lb.	Per Ct.
20	40	13.10	0.912	55.3	—	—	—
23	31	—	—	—	16.31	0.720	66.5
17	23.3	29.77	0.878	55.0	24.14	0.910	72.0
22	20.0	22.71	0.863	60.0	24.65	0.696	72.0
37	17.5	45.15	0.775	62.5	39.90	0.694	72.6
27	15.0	44.65	0.444	64.0	48.25	0.555	72.4
8	12.5	15.70	0.503	64.0	15.70	0.264	66.0
11	10.0	17.80	0.456	77.0	17.90	0.138	55.2

pound of volatile matter destroyed, began to decrease in the low temperature tank with a digestion time below 20 days and in the high temperature tank below 17.5 days. At the same time the apparent volatile matter destruction, as indicated by loss of weight, based on analyses, did not decrease at all or at a much slower rate. During the first period of the experiment (154 days) the total gas production in the low temperature tank amounted to 1170.1 cubic feet, with a volatile matter addition of 124.2 pounds. The total weight of the gas was 60.68 pounds while 68.3 pounds of volatile matter were destroyed, corresponding to 0.89 pounds of gas per pound of volatile matter destroyed. This corresponds well with the results obtained in the second period, when the digestion time was decreased to 20 days. After the time of digestion had been shortened to a period of 10 days, excessive foaming made it necessary to let the tanks rest. After resting, with additions of 27.1 pounds of volatile matter during a period of 18 days (digestion schedule 17.5 days) gas production amounted to 20.25 cubic feet per pound of

volatile matter added, as compared with 8.83 cu. ft. gas produced per pound of volatile matter during the 112 days on a 40-day digestion schedule. This large amount of gas undoubtedly came from the previously liquefied material. It appears therefore, that the two successive stages of digestion (liquefaction followed by gasification) can be modified by overloading and that the fundamental phenomena observed in digestion of fresh solids alone holds as well for properly and improperly seeded material. It is also evident that drastic reduction in digestion time must of necessity result in less production of gas. The apparent reduction in volatile matter, as measured by standard methods of analysis, is not a true indication of the destruction of volatile matter to its end products. Liquefied material would be discharged in the overflow in the case of separate digestion tanks, or pass into the flowing-through chamber of an Imhoff tank, thus putting a heavier load upon subsequent oxidation devices.

Several drainability tests were made on samples of sludge drawn from both tanks in the laboratory and also in the field. The field tests

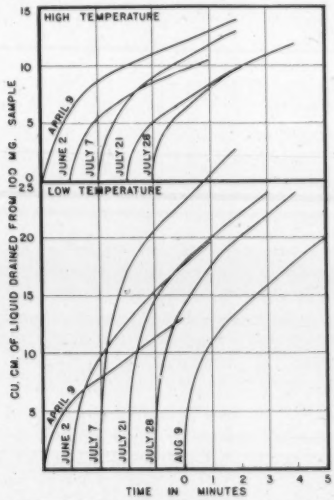

FIG. 6.—Drainability of sludge drawn from low and high temperature tanks.

consisted of pouring the sludge on sections of the open sand bed used for sludge drying at the Freehold treatment plant. The low tempera-

ture sludge drained more rapidly than the thermophilic, and surface cracks formed sooner. The surface of the high temperature sludge remained intact, similar to the drying of a gelatinous mass. In the laboratory tests 100 grams of sludge was placed on top of coarse filter paper in 3¼ inch Buchner funnels and suction maintained at 22 inches. The amounts of filtrate collected in 5 minutes for some of the tests are shown in Figure 6. The material used had the following solids and ash contents:

Results in Per Cent

Date	Low Temperature			High Temperature		
	Solids	Ash	Liquid in 5 Min.	Solids	Ash	Liquid in 5 Min.
4/9	5.15	42.0	13	6.63	46.2	12
6/2	4.53	—	20	5.98	43.3	11
7/7	4.07	37.7	28	6.16	41.6	13
7/21	4.95	40.5	24	4.96	39.5	10
8/9	2.69	33.2	24	1.63	35.7	12

The amounts of water drained from the low temperature sludge were materially greater than from the high. In those cases where the percentage solids concentration in the high temperature sludge were from 25 to 35 per cent greater than in the low, the relative amounts of water drained remained about the same.

Discussion

Seeding the high temperature tank with digesting sludge obtained from the local separate digestion unit, which was maintained at 80 to 85° F., failed to produce proper thermophilic sludge even after several months of operation. It appears that thermophilic sludge could be produced only from fresh solids added and kept at a temperature of 120 to 125° F. The failure to produce active thermophilic sludge caused a number of operation troubles such as scum formation and little or no production of gas.

The insulation of two inches of asbestos was insufficient to prevent great heat losses, so that during the coldest time of the year the higher temperature could not be maintained. More detailed information on heat losses will be published subsequently.

The shortest practical time for digestion in the low temperature tank was 20 days, and under existing conditions in the high temperature tank 17.5 days. It should be remembered that true thermophilic digestion was not established during the period of operation under dis-

cussion. The general results indicate that nothing is gained, and as a matter of fact troubles are invited, when material digesting under mesophilic conditions is heated too high. When the temperature is raised and maintained for a considerable time at a higher level, the flora responsible for mesophilic digestion are gradually changed to an intermediate stage and neither thermophilic nor mesophilic digestion is accomplished. The results show also that raising the temperature temporarily does not interfere with gas production or other general digestion activities, except that the sludge produced may drain more poorly and odors may become more pronounced. Under such conditions the sludge drawn and placed on drying beds will not show surface cracks, so that air cannot enter to dry the material inside and all drying must proceed by surface evaporation. The dried surface crust keeps the inner material moist for a much longer time.

From the results obtained it is evident that, as a rule, the concentration of the material in the high temperature tank was greater than in the low. During the first period the scum blanket was far more pronounced in the high temperature tank. It appears essential that adequate provision be made for mixing the tank contents to allow the escape of gas, obtain proper digestion of the concentrated mass and prevent clogging of lines. When the digestion time was reduced in the low temperature tank somewhat similar conditions were encountered. With smaller digestion capacities proper stirring would be advantageous. The fact that at some places stirring and mixing have been found beneficial, while at other places they have not, can probably be explained on this basis. There is no question that, with the help of stirring and mixing, gas production became more uniform. More detailed results on this phase of the work will be given in a subsequent paper.

Although the supernatant liquor contained about the same concentration of solids in both tanks, the ash content of the solids in the liquid of the high temperature tank was lower. The black colored supernatant of the high temperature tank had an intensely disagreeable odor, especially during the first period. This odor, apparently caused by liquefied intermediate decomposition products of a nitrogenous character, gradually changed during the second period to a pungent ammonia odor. In this respect it is interesting to note that the pH values never fell below 7.0 in either of the tanks, and fluctuated between 7.9 to 8.2 in the high temperature tank.

Summary

Experiments conducted at Freehold, New Jersey, in two pilot digestion tanks each of 140 gallons capacity showed that attempts to produce thermophilic sludge from digesting mesophilic sludge, to which fresh

solids were added, failed. Operation difficulties, scum formation, objectionable odors and little gas production resulted. Drainability of the sludge drawn was poor. The digestion periods of sludges kept at 80 to 85° F. and 120 to 125° F., were gradually reduced from 40 to 10 days. After the period in the low temperature tank was reduced to less than 20 days, gas production decreased and finally excessive foaming resulted. Similar conditions were found when the period in the high temperature tank was reduced to 17.5 days. No true thermophilic digestion was obtained. The volume and weight of the gas decreased, while liquefaction increased rapidly below these minimum digestion schedules, but the apparent volatile matter reduction, as measured by standard laboratory methods, remained constant. These usual measurements are known to be unreliable. Gas production from the low temperature material varied from 0.91 lb. per lb. volatile matter destroyed, with the longer digestion schedule, to 0.456 lb. with the shortest, and from 0.910 to 0.138 lb. for material in the high temperature tank.

Acknowledgment

The authors desire to express their sincere appreciation to Mr. W. B. Denise, plant operator, whose experience made him invaluable as the operator of the experimental tanks, and to Mr. W. J. Schiverea, Borough Engineer, for his help in building the tanks and for following the work with great interest.

A COLORIMETRIC METHOD FOR THE DETERMINATION OF DISSOLVED OXYGEN *

By Moses L. Isaacs

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There are many problems which require the determination of dissolved oxygen under conditions which make the use of the standard volumetric method difficult. These occur particularly in the field and in the small laboratory where the reagents and apparatus needed may prove too complicated or cumbersome for convenient use.

Several methods based on colorimetric procedures have been proposed for simplifying the technique of the analysis. The earliest of these was proposed by Ramsay and Hofray, who used cuprous chloride as the test reagent. Rideal at the same time suggested the colorimetric estimation of the iodine set free in the volumetric method. Winkler proposed the use of adurol, a photographic developer. None of these methods have found favor in practice either because of difficulties in the preparation of standards or the reagents involved. More recently a method has been proposed by McCrumb and Kenny which is intended for small quantities of oxygen present in boiler water. It resembles the volumetric method in many respects except that in the final stage the iodine is determined colorimetrically.

In the present paper a method is described which employs amidol (di-aminophenol hydrochloride). This colorless substance is easily oxidized by free oxygen with the formation of one or more colored compounds. The rate of oxidation is progressively more rapid the more alkaline the medium but at the same time the depth of color as well as the shade becomes more variable and difficult to reproduce. In an acid medium, on the other hand, the color development is less rapid but much more uniform. For a quantitative method it is therefore necessary to choose a degree of acidity which represents a compromise between rapidity and reproducibility of color formation. At a pH of about 5.1 the color is practically at a maximum in 30 minutes and for the attainment of this pH it is convenient to use the acid of the amidol with potassium citrate, a combination which produces a powerful buffer action. The quantity of amidol chosen as a basis for the method was two grains (0.13 grams) since this quantity is available on the market in tablet form.

* Presented at the Meeting of the New York State Sewage Works Association, New York City, January 15, 1935.

The color may be measured in a number of ways as for example by comparison with a set of permanent standards or by dilution of a given volume of standard until a match is obtained with the unknown. In the development of the method the Dubosque Colorimeter was used with excellent results.

The details of the method as finally determined are as follows:

Apparatus Needed

1. Test tubes, glass stoppered, inside diameter 14 mm., length 150 mm. Instead of these, Pyrex test tubes, No. 2370 (outside dimensions 16 by 150), provided with No. 0 rubber stoppers may be used. For greater accuracy it is well to check the internal diameter and discard tubes which vary more than 0.2 mm. from 14 mm. The volume of these tubes is approximately 24 ml.

2. Comparator block * containing 6 holes, arranged in two rows, (Walpole Comparator) and provided with a back consisting of a frosted glass and yellow screen. The screen may be most conveniently made of two layers of No. 300 amber cellophane or of a 3 cm. thickness of 2 per cent potassium chromate. One layer of light green cellophane and one layer of amber makes a better screen for some observers.

3. Pipette, 0.5 ml., for citrate solution.

4. Graduated test tube (14 mm. inside diameter). Graduated from 1 to 20 ml. in 0.5 ml. divisions.

5. Pipette or medicine dropper for diluting standard.

6. Bottles for permanent standard and reagents (2 to 4 needed).

In place of items 4 and 5 a set of permanent standards in test tubes or the Dubosque Colorimeter may be used.

Chemicals and Solutions Needed

1. Amidol. Any of the following may be used:
   - *A.* "Tabloid" amidol, 2 grain tablets.†
   - *B.* Amidol crystals. The product should be of a light gray or tan color.
   - *C.* Amidol solution. A solution of 1.3 grams of amidol in 10 ml. of water. This must be freshly prepared.
2. Potassium citrate.‡

* The glass stoppered tubes and the comparator block may be obtained from Eimer and Amend, New York City.

† Tabloid products may be obtained from the Burroughs Wellcome Co., Tuckahoe, N. Y.

‡ A solution of sodium citrate (20 gms. dissolved in water and diluted to 100 ml.) may be used. One ml. of this is used in the test.

A solution of 72 grams dissolved in water and made up to 100 ml.

3. Permanent standard which contains:

Cobalt chloride, $CoCl_2 \cdot 6H_2O$	
1 M solution (238.0 gms. per liter)	50.0 ml.
Potassium dichromate, $K_2Cr_2O_7$	
1/20 M solution (14.7 gms. per liter)	1.3 ml.

diluted to 100 ml. with distilled water. This solution has the color equivalent to 15 p.p.m. of dissolved oxygen.

4. Distilled water.

Procedure

To the sample of water, which should completely fill the test tube, add 0.5 ml. of the citrate solution and 0.13 grams (2 grains) of amidol (or 1 ml. of the solution). Insert the stopper quickly, avoiding the entrainment of air bubbles. Mix thoroughly by inverting a number of times and allow to stand at least one half hour. At the end of this time make comparison with the permanent standard. The color increases slowly but readings may be made for at least two hours.

With permanent standards in tubes or with the dilution of strong standard the comparator is used. In the latter case a convenient quantity of cobalt standard is placed in the graduated test tube. Water is added until the color matches that of the unknown. The following formula then applies:

$$x = 15 \frac{\text{Original volume of permanent standard}}{\text{Final volume}}$$

where x represents the parts per million of oxygen in the solution tested.

Turbidities up to 900 p.p.m. may be compensated for by the use of untreated water placed behind the cobalt standard.

In diluting the cobalt standards corresponding to 8 p.p.m., differences of 0.5 p.p.m. may be noted, while in dilutions of about 2 p.p.m. differences of less than 0.10 p.p.m. may be determined.

The Dubosque Colorimeter, where available, may be used with good results. The test tube containing the unknown is placed diagonally across one of the cup holders. The cobalt standard is placed in a cup which is placed in the second cup holder. The colored screen is placed across the top of the eye piece. The equation is $x = 1.07 \times$ depth of the cobalt standard, where $x =$ the number of p.p.m. of dissolved oxygen. With this method, readings may be made which give results differing in the higher concentrations by not more than 5 per cent.

A correction for the dissolved oxygen in reagents may, in most cases, be neglected. Examination of a number of amidol tablets has shown that

they contain approximately 0.003 mg. of oxygen per tablet which involves a correction in the 24 ml. of solution of a little over 0.1 p.p.m. The potassium citrate solution is almost free from dissolved oxygen. This solution therefore dilutes the oxygen content of a sample. When the latter contains 8 p.p.m. the correction would be 0.16 p.p.m. With waters containing smaller oxygen contents the correction decreases and becomes practically zero when the solution contains no dissolved oxygen.

Temperatures between zero and 30 degrees centigrade do not influence the amount of color formation at the end of the 30-minute period.

The Effect of Other Substances in Water

As is well known there are present in many waters oxidizing or reducing substances which are likely to affect the results obtained by the method. Of the oxidizing agents, the quantities tolerated without interference are: nitrites (expressed as nitrogen), 2 p.p.m.; iron (ferric), 10 p.p.m.; chlorine, 2 p.p.m. Of the reducing substances 5 p.p.m. of sodium sulphite may be tolerated and 2 p.p.m. of hydrogen sulphide.* These figures indicate that the method is most sensitive to reducing agents and least sensitive to oxidizing agents. Of the substances mentioned above, nitrite will probably prove the most troublesome. Fortunately the presence of nitrite in a solution from which other oxidizing agents are absent may be demonstrated qualitatively by the use of amidol alone. In the presence of nitrite a deep reddish brown color develops quickly and grows progressively deeper. The omission of the citrate makes the medium more acid and the action with dissolved oxygen is therefore slow.

Sodium chloride in concentrations below 0.5 per cent does not interfere with the test but with higher concentrations the readings are too high. The method, therefore, cannot be used under conditions which involve the presence of brine or sea water. Where the salt concentration is constant it should be possible to make a set of standards proportional to the oxygen concentration.

Discussion

The amidol method, while suffering from a number of limitations, is presented at this time in the hope that it may find application in the study of some of the problems involving dissolved oxygen. Included among these are field studies, determinations of biological oxygen de-

* These tolerances differ somewhat from those given by Mr. Gilcreas in the discussion which follows this paper. The explanation for this probably lies in the use of different depths which cause interfering colors to affect the test at different concentrations.

mand and comparative studies of oxygen content in a given water supply. The simplicity of the procedure and the portability of the apparatus make the test particularly adapted for use in the small laboratory and for field work.

The method as given, however, should not be considered as necessarily in its final and most usable form. In the discussion following this paper Mr. Gilcreas has presented some excellent modifications which should make the method of more practical use in the field. Further investigations may permit the use of a more alkaline medium which would give a more rapid color development and would considerably reduce the interference of nitrites and other similar substances. Another desideratum is a procedure whereby the color when once developed could be stabilized so that a solution might be poured from one vessel into another without fear of increasing the color by further absorption of oxygen.

In order to facilitate further experimentation a few points of interest concerning the color reaction may be given, although the chemistry of the reaction is unknown. It is probable that a number of chemical compounds may be formed. Practically every color of the spectrum may be obtained by varying the pH of the reaction. At a pH of 5.1 at the end of the thirty minute period there are present at least two substances, one red and one yellow. The concentration of the red substance is proportional to the original concentration of dissolved oxygen while that of the yellow substance is not. The purpose of the yellow screen is to rule out the effect of the latter compound. An important characteristic of the red compound is that it represents an intermediate product in the complete oxidation of amidol. If a solution of it is exposed to the air there is formed a black insoluble substance and the red color disappears. This black substance, although insoluble in water, is soluble in strong acids and somewhat soluble in alcohol. In each case the solution is colored red. Such a solution has possibilities for use in the preparation of permanent standards although at the present time it appears to offer no advantage over the cobalt standard described above.

Summary

A method for the colorimetric determination of dissolved oxygen is described which uses amidol and potassium citrate. The action is carried out in test tubes and comparison is made with permanent standards made of solutions of cobalt chloride and potassium dichromate. The colorimetric determination may be made with a Dubosque Colorimeter or with a comparator block, using tubes of permanent standards, or by diluting a given quantity of the permanent standard.

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DISCUSSION

BY F. W. GILCREAS

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The amidol test for the colorimetric estimation of dissolved oxygen, which Dr. Isaacs has developed, provides a simple and rapid test in place of the more complicated Winkler method. Such a test has long been needed for use in field studies and in small plant laboratories where equipment and trained personnel are not available.

Dr. Isaacs very kindly supplied the details of the method, which enabled the Division of Laboratories and Research of the New York State Department of Health to make a preliminary study of the possibilities and limitations of the procedure. Experience indicated that the 14 mm. test tubes suggested by Dr. Isaacs are awkward to handle and unsatisfactory for the collection of representative samples. The short depth of solution does not permit a sharp differentiation of color due to small increments of oxygen. As a measure of the oxygen concentration, the dilution of a standard in a graduated tube to a color matching the unknown sample seems inconvenient for field work and it was also found that this method was not especially accurate except in the hands of an experienced person. Therefore, the technic as outlined by Dr. Isaacs was modified for the use of glass-stoppered, two-ounce, French square bottles in place of the 14 mm. test tubes. These bottles are, of course, bulkier than the tubes but they are easy to handle, less fragile, and permit the collection of satisfactory samples in accordance with the standard technic. The cross-section of the bottles is 30 mm. and this greater depth of color provides a closer interpolation of changes in color due to small increments of oxygen. The entire study was made with these bottles and they were found suitable for the purpose.

The greater depth of color necessitated a modification of the standard proposed by Dr. Isaacs. In order to avoid the use of the yellow screen in viewing colors, the method was modified by the addition of definite quantities of potassium dichromate solution to produce the proper amount of yellow in the standard. Table I gives the quantities of cobalt chloride and potassium dichromate required for producing standards equivalent to given amounts of oxygen. The colors can be viewed best

against the sky, particularly against north light. Direct sunlight is unsatisfactory.

TABLE I.—*Preparation of Permanent Standards*

(The amounts of cobalt chloride and potassium dichromate solutions given are diluted to 50 cc. with distilled water)

Oxygen	Cobalt Solution	Potassium Dichromate
p.p.m.	c.c.	c.c.
0.0	0.3	0.5
1.0	0.6	1.1
2.0	1.5	2.0
4.0	3.8	3.1
6.0	6.8	4.6
8.0	10.4	6.0
10.0	14.4	6.6
12.0	19.5	5.1
14.0	30.0	3.4

Cobalt chloride solution: 476 grams of cobalt chloride and 100 c.c. of concentrated hydrochloric acid dissolved in water to make one liter. The solution usually requires filtration.

Potassium dichromate solution: 2.5 grams of potassium dichromate and 20 c.c. of concentrated hydrochloric acid dissolved in distilled water to make one liter.

A number of experiments gave satisfactory results with the standards, permitting accurate determinations of oxygen as compared with the Winkler method. Time did not permit an extended investigation of the sensitivity of the method. However, the results indicated that an accuracy of 0.5 p.p.m. could readily be obtained throughout the entire range of colors corresponding to oxygen concentrations from zero to 14 p.p.m. In the lower ranges, that is with concentrations less than 6 p.p.m., the colors can be matched with an accuracy of at least 0.2 p.p.m. The bottles can be used in a comparator, making possible the use of the double-bottle method to compensate for the presence of color and turbidity in the sample. It is probable that turbidities greater than 100 p.p.m. will interfere with the determination when the two-ounce bottles are used.

Studies indicated that the color produced by the amidol does not develop immediately but gradually changes during a period of thirty minutes from a yellowish brown to a decidedly reddish brown. At the end of thirty minutes, the color becomes permanent and so remains for at least two hours. Exposure to light and variations in temperature produce only insignificant changes in the color developed. Precision in the addition of reagents is not required. It was found that the amounts of citrate and amidol can be varied from one-half to twice the recommended quantities without affecting the color produced, which is an advantage in field work.

The solutions of amidol and sodium citrate were found satisfactory for the test and stable for relatively long periods. The tablets of amidol suggested by Dr. Isaacs worked well but they tend to discolor after the tubes are opened, owing to oxidation of the amidol. This oxidized material is not readily soluble but remains as a precipitate, interfering with the reading of the colors when bottles are used. Mr. Kelly of this laboratory suggested the use of ordinary, druggists' No. 2, gelatin capsules for holding solid amidol. When completely filled, these capsules contain 0.26 grams of amidol with a possible variation of $\pm$ 0.02 grams, and oxidation is avoided. Powdered sodium citrate can also be used in similar capsules. The possible error due to the introduction of entrained air by tablets of the reagents is a serious disadvantage. Since the tablets sink to the bottom of the sample, any entrained oxygen is dissolved in the sample. The use of the capsules avoids such a possible error as the loosely packed powder first settles on the surface of the sample, discharging any entrained air without the possibility of its being dissolved and thus increasing the dissolved oxygen content. For the proper development of the color, the solutions must be heavily buffered prior to the reaction of the amidol with oxygen. Hence the citrate must be added and dissolved before the addition of the amidol.

No extensive study of the method was possible in the time given to this work. However, the effect of certain substances which interfere with the Winkler method was investigated. Oxidizing agents produce but little effect on the amidol color in the quantities in which they would normally be present in water and sewage. Chlorine in amounts greater than 1.5 p.p.m. produces offshades, the interference increasing with larger amounts of chlorine. Ferric iron does not interfere seriously unless present in amounts greater than 5 p.p.m. Reducing substances have a more serious effect on the reaction. Interfering colors with serious offshades result when the concentration of ferrous iron and sulphites exceeds 3 p.p.m., when nitrites are greater than 1 p.p.m., and sulphides in excess of 5 p.p.m. Such concentrations would frequently be encountered in sewage and heavily polluted streams. It is possible that the method can be modified to permit its use in the presence of these substances.

Comparative determinations with the Winkler method were conducted, in which various types of water as received for routine examination were used. Excellent agreement was obtained. The method for the determination of biochemical oxygen demand was also studied and the variation from the standard procedure found to be less than 10 per cent. Organic matter and other substances present in the sewage do not affect the reaction. The incubation of the samples for the determi-

nation of biochemical oxygen demand was carried out in the two-ounce bottles with satisfactory results.

Our brief experience with this method indicates that it should prove a valuable addition to the methods of water and sewage analysis. Much less manipulation, laboratory equipment, and skill are required than with the standard modified Winkler method. Two reagents in place of six are necessary and interfering action due to substances normally present in water is less than with the standard method. Permanent standards can be readily prepared and would seem to give satisfactory results. Apparently the use of tablets or capsules for the addition of the reagent will prove suitable and preclude the necessity of preparing and transporting solutions of reagents. Further study is required to determine the limitations of the method and the precision to be expected from it. Presumably, it will not be as precise a measure of the dissolved oxygen as is the standard Winkler method. However, the ease and rapidity with which it can be carried out will undoubtedly render it valuable for use in small plant laboratories and for field work. Additional study of the method and of the standards may indicate an even greater accuracy in reading than 0.5 p.p.m. found by the present technic. The thirty-minute period necessary for full development of the color is inconvenient for field work, but this is not a serious objection when compared with the time required for the standard method.

Plant Operation

STANDARD METHODS OF SEWAGE ANALYSIS

Report of the Committee on Methods of Sewage Analysis of the Federation of Sewage Works Associations

W. D. HATFIELD, *Chairman*

Members	*Associate Members*	
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INTRODUCTION

The Committee on Standard Methods of Sewage Analysis has attempted to present the best available method for each determination, basing its choice as much as possible on the following factors; accuracy, simplicity, interfering substances, and general use throughout the United States by State Boards of Health, sewage and research experiment stations and laboratories. In some cases, two or more procedures are given because of the interfering substances or because the methods are in about equal use throughout the country. In two cases, references are given to procedures in the 7th Edition of Standard Methods of Water Analysis (A. P. H. A.) that were not deemed of sufficient value to print in this set, but still were worth mentioning.

The methods are written primarily for the use of chemists and not with the idea that the procedure will be sufficiently detailed for use by the untrained man. However, with competent supervision, the non-chemically trained man may become quite proficient with any of these methods. An attempt has been made, with the limited space available, to call attention to the interfering substances and inaccuracies of most of the methods.

The text has been divided into two parts: the examination of sewage effluents, industrial wastes and grossly polluted waters; and the examination of sewage sludges and muds. The determinations have been numbered with Roman numerals, the preparation of reagents and procedures with Arabic numerals and the reagents by letters.

In the preparaiton of reagents the distilled water must not contain substances which will interfere with the particular determination. All

chemicals used shall be of c.p. reagent quality unless otherwise specified in the text. The abbreviation ml. for milliliter has been used throughout the text instead of cc.

Insofar as possible, standard laboratory glassware and apparatus have been specified. For certain work special apparatus may greatly simplify the technic or the time required for the determination. The use of special apparatus is not precluded but encouraged where it has been proved to give equal or better results than the standard apparatus.

It has also been the purpose to minimize, as much as possible, the number of standard solutions which a laboratory must keep on hand. Therefore, special standard solutions which read the p.p.m. direct have not always been recommended if other standard solutions are available which can be used with a factor. This does not mean that such direct reading standard solutions cannot be used.

The Chairman of this committee is also a member of the editorial committee for the next edition of Standard Methods of Water and Sewage Analysis of the American Public Health Association. This report of your committee of the Federated Sewage Works Association will undoubtedly be accepted, with minor modifications if found desirable, as the standard methods for the sewage section of the next Standard Methods of Water and Sewage Analysis (A. P. H. A.). Your chairman, therefore, urges that all criticisms of methods, style, additions, deletions, etc., be sent to him at the earliest possible date. (249 Linden Ave., Decatur, Illinois.)

W. D. HATFIELD, *Chairman*

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PART I. EXAMINATION OF SEWAGE, EFFLUENTS, INDUSTRIAL WASTES, AND GROSSLY POLLUTED WATERS

I. COLLECTION OF SAMPLES

1. *Representative Samples*

Care shall be taken that samples are representative. The great variety of conditions under which collections must be made renders it impossible to prescribe a fixed procedure.

2. *Methods of Sampling Sewage and Effluents*

The standard method comprises collection of composite samples over two 12- or one 24-hour period. Individual portions should be taken in a wide-mouthed bottle having a diameter at the mouth of at least 25 mm. and a capacity of at least 120 ml. These should be collected each hour, in some cases each half hour, or even every five minutes, and mixed at the end of the sampling period or combined in a single bottle as collected (see preservation instructions below). It is desirable, *in many cases absolutely essential, to combine the individual samples in volumes proportionate to the volume of flow.* Final volume of 2 to 3 liters is sufficient for sewage and effluents. Automatic sampling devices are available, but should be checked periodically against standard sampling, particularly on oxidized effluents where oxidizing growth accumulates. They should be cleaned daily of all growths of sewage organisms.

3. *Preservation of Samples*

Samples of sewage and unstable effluents should be analyzed as soon as possible after collection. No single method of preservation is entirely satisfactory, and the preservative should be chosen with due regard to the determinations that are to be made.

Samples for the determination of relative stability and biochemical oxygen demand must be free from all preservatives. When samples are composited, the individual samples or the composited sample should be kept in a cooler at 4 to 10° C. during the compositing period. When the sample cannot be analyzed within 4 to 6 hours after collection, or after the compositing period (24 hrs.) it should be preserved as follows:

1. Add 5 ml. of washed chloroform per liter and store on ice or at 4° C. The chloroform should be washed in a separatory funnel three times with equal volumes of distilled water and then kept dry by storing it in bottles containing anhydrous calcium chloride.[1]

2. Sufficient sulphuric acid (C.P.) to produce 1500 p.p.m. acidity will preserve the nitrogen balance. This treatment lowers the suspended matter, settlable matter, and oxygen consumed values.

Note: Formaldehyde affects so many of the determinations that its use is not recommended.

II. Physical Examination

The *temperature* should be recorded at the time of sampling to the nearest degree centigrade. *Turbidity* may be determined by using a Jackson candle turbidimeter provided the suspended matter is finely divided. If the suspended matter interferes, the turbidity may be run on the settled sample. The results should be reported on settled or unsettled samples. The *color* of sewage is largely dependent on industrial wastes and the age of the sewage. This determination should be descriptive of the color and intensity, but is not at present on a quantitative basis.

A considerable advance has been made in recent years in the quantitative determination and expression of *odors* in waters.[2, 3] These methods have not been applied to sewage, but it is suggested that the threshold odor be determined with a "Fair" type osmoscope by the water dilution method; 100 ml. of the sample is diluted with 100 ml. of odor-free distilled water (having been treated with activated carbon), in a 500 ml. Erlenmeyer flask. 100 ml. of this mixture is diluted with 100 ml. of odor-free water; 100 ml. of this mixture, etc. Beginning at the highest dilution, each dilution is shaken and "sniffed" through the osmoscope. That dilution at which the odor is just detectable is the threshold odor. The intensity of odor may be expressed as follows:

(1) pO = the number of 100 per cent dilutions made to obtain the threshold odor,

or

(2) The dilution ratio to obtain the threshold odor.

100% Dilutions	pO	Dilution	Dilution Ratio	
0	0	0	0	$2^0=0$
1	1	1 : 1	1	$2^1=1$
2	2	1 : 4	4	$2^2=4$
3	3	1 : 8	8	$2^3=8$
4	4	1 : 16	16	$2^4=16$
5	5	1 : 32	32	$2^5=32$
etc.	etc.	etc.	etc.	etc.

$$pO = \frac{\text{Log dilution ratio}}{\text{Log } 2} = \frac{\text{Log Dil. Ratio}}{.3010}$$

Thus the pO of a 1 to 200 dilution is

$$pO = \frac{\log\ 200}{.3010} = \frac{2.3010}{.3010} = 7.64$$

also 2^{pO} = dilution ratio, or if $pO = 7$, $2^7 = 128$ or the dilution ratio.

III. Expression of Chemical Results

Analytical results shall be expressed in parts per million by weight, assuming that 1 liter of water, sewage or industrial waste weighs 1 kilogram. Industrial wastes of high specific gravity shall have the results expressed as milligrams per liter with the specific gravity given. Only the significant figures should be recorded.

In reporting results on stream pollution or plant operation and efficiencies, it is desirable to express the results on a weighted basis, thus including both the concentration and the volume of flow. These weighted results may be expressed,[4] as Quantity Units after the U. S. Public Health Service, as pounds per 24 hours, or as population equivalents (based on B.O.D.). Totals of the weighted units may be converted to the weighted average parts per million.

Calculations:

$$\text{Quantity Units}_1 = \text{p.p.m.} \times 1000 \text{ cu. ft. per second}$$

$$\text{Quantity Units}_2 = \text{p.p.m.} \times \text{million gallons per day (m.g.d.)}$$

$$\text{Pounds per 24 hours} = \text{p.p.m.} \times \text{m.g.d.} \times 8.34$$

$$\text{Population equivalent} = \text{p.p.m. 5-day B.O.D.} \times \text{m.g.d.} \times \frac{8.34}{0.17}$$

IV. Ammonia Nitrogen

Because organic nitrogen in unsterilized sewage is progressively ammonified by saprophytic bacteria, the determination of ammonia or organic nitrogen must be made as soon as possible after collection of the sample, or the sample must be preserved with an excess of 1500 p.p.m. sulphuric acid.

Direct Nesslerization is preferable to distillation because it saves time.

DIRECT NESSLERIZATION [1]

1. *Reagents*

(*a*) Copper sulphate. Dissolve 100 g. copper sulphate in ammonia-free distilled water and make up to one liter.

(*b*) Lead acetate. Dissolve 100 g. of lead acetate in ammonia-free distilled water and make up to one liter.

(*c*) Zinc sulphate. Saturated solution in ammonia-free distilled water. (542 gm. $ZnSO_4$ per liter).

(*d*) Sodium hydroxide. Dissolve 500 g. of sodium hydroxide in ammonia-free distilled water and make up to one liter.

(*e*) Ammonia-free water.

1. Ammonia-free water may be made by redistillation of distilled water that has been treated with bromine and allowed to stand over night.

2. For most work the ammonia can be removed from ordinary distilled water by shaking with Folin's ammonia permutit. However, traces of magnesium in some distilled water causes cloudy Nessler tubes. Cloudy tubes may be prevented by the use of solution (*f*).

3. Shake 1 gal. of tap water with 10 grams of Folin's ammonia permutit (the permutit may be used repeatedly) to absorb the ammonia; decant and precipitate calcium and magnesium with 25 ml. of 33 per cent NaOH solution (*d*) and 8 ml. of $CusO_4$ solution (*a*), shake, settle over night and decant the clear liquor. This ammonia- and magnesium-free water does not give cloudy tubes on Nesslerization, and can be used in all steps of the Nesslerization process, but not being a distilled water it cannot be used for making up reagents.

(*f*) Rochelle salt solution. Dissolve 500 g. c.p. Rochelle salts in 1 liter of water.

(*g*) Standard Ammonium Chloride. For the stock solution dissolve 3.818 g. of ammonium chloride in 1 liter of ammonia-free distilled water. Dilute 10 ml. of this stock solution to 1 liter with ammonia-free distilled water; 1 ml. contains 0.01 mg. of nitrogen.

(*h*) Nessler Reagent. Dissolve 50 g. of potassium iodide in 35 ml. of cold ammonia-free distilled water. Add a saturated solution of mer-

curic chloride, made with ammonia-free water, until a slight precipitate persists (about 415–20 ml.). Add 400 ml. of 9 N sodium hydroxide (360.09 g. of NaOH in 1 liter of ammonia-free water). Dilute the solution to 1 liter, allow it to clarify by standing a few days and decant.

2. *Procedure*

Add 1 ml. of copper sulphate solution (*a*) to 100 ml. of the sample in a 120 ml. bottle or tube; mix thoroughly, add 1 ml. of sodium hydroxide solution (*d*), again mix and clarify by settling or centrifuging. If settling is not satisfactory, repeat, adding the sodium hydroxide first and the copper sulphate last.

If the sample contains hydrogen sulphide, substitute 1 ml. of lead acetate solution (*b*) or zinc sulphate solution (*c*) for the copper sulphate solution in the above procedure.

Dilute 5 ml. or less of the clarified supernatant liquor to the mark in a 50 ml. Nessler tube with ammonia-free water, add 1 or 2 drops of Rochelle salt solution (*f*) to prevent cloudy tubes, Nesslerize by adding 1 ml. of Nessler's reagent and after 10 minutes compare with standards prepared as follows:

Make up a series of ammonia standards in 50 ml. Nessler tubes by diluting 0.2, 0.4, 0.7, 1.0, 1.4, 1.7, 2.0 and 2.5 ml. of ammonium chloride standard (*g*) to the mark with ammonia-free water. Add 1 ml. of Nessler reagent (*h*) to each tube and allow ten minutes for color formation.

PREPARATION OF PERMANENT STANDARDS

1. *Reagents*

(*a*) Potassium chloroplatinate K_2PtCl_6. Dissolve 2.00 g. of the salt in a small volume of distilled water, add 100 ml. of concentrated hydrochloric acid, and dilute to a liter with distilled water.

(*b*) Cobaltous chloride $CoCl_2 \cdot 6H_2O$. Dissolve 12 g. of dry crystals in a small volume of distilled water, add 100 ml. of concentrated hydrochloric acid, and dilute to 1 liter.

2. *Procedure*

Measure into 50 ml. Nessler tubes the volumes of these solutions indicated in the table, and dilute to the mark.

The values given in the table are approximate: actual equivalents of the standards thus prepared will differ with the quality of the Nessler reagent and the color sensitiveness of the analyst's eye. They should be compared with Nesslerized ammonia standards and the tints modified as necessary. Such comparison should be made for each newly prepared Nessler solution and checked by each analyst.

Preparation of Permanent Standards for the Determination of Ammonia

Value in Ammonia Nitrogen	Volume of Platinum Solution	Volume of Cobalt Solution
mg.	ml.	ml.
0.000	1.2	0.0
.001	1.8	.0
.002	2.8	.0
.004	4.7	.1
.007	5.9	.2
.010	7.7	.5
.014	9.9	1.1
.017	11.4	1.7
.020	12.7	2.2

These standards may be kept for several months if stoppered and protected from dust. They require no addition of reagent. Comparison with them is made at least 10 minutes after the distillate has been Nesslerized.

Calculations:

$$\frac{\text{ml. } NH_4Cl \text{ standard}}{\text{ml. sample Nesslerized}} \times 10 = \text{p.p.m. ammonia nitrogen}$$

or

$$\frac{\text{mg. of N in standard}}{\text{ml. sample Nesslerized}} \times 1{,}000 = \text{p.p.m. ammonia nitrogen}$$

Note: A correction for the reagents added for clarification of the sample is not necessary unless larger amounts are used than directed.

DISTILLATION

3. *Reagents*

(*a*) M/2 Phosphate buffer solution. Dissolve 14.3 gms. of monobasic potassium phosphate (KH_2PO_4) and 90.15 gms. of dibasic potassium phosphate ($K_2HPO_4 \cdot 3H_2O$) in ammonia-free distilled water and make up to one liter.

4. *Procedure* [5, 6]

Steam out of the still until free from ammonia. Place the sample (10 to 100 ml.) in an 800 ml. Kjeldahl flask. Add 25 ml. of the M/2 phosphate buffer solution (*a*) which keeps the pH of the distillation mixture at pH 7.4 during the distillation, dilute with 300 ml. of ammonia-free water and distill over 200 ml. Dilute an aliquot portion of the distillate with ammonia-free water to 50 ml. in a Nessler tube, Nesslerize and compare with standards as directed Sec. IV, p. 450.

Calculation:

If R = reading of ammonia standard in ml. NH_4Cl standard solution,
s = ml. of sample taken for distillation,
d = ml. of distillate taken for Nesslerization,

then $\frac{2000}{s \times d} \times R$ = p.p.m. ammonia nitrogen.

V. Albuminoid Nitrogen

The ratio of albuminoid nitrogen to total nitrogen is quite variable in sewage [7, 8, 9] due to variable amounts of urea [10] decomposed. Most laboratories have therefore discontinued this determination. A few have continued it in order to correlate their results with those of past years. If the determination is required, it may be made as directed in Pt. I, Sec. VIII, Standard Methods of Water Analysis, Edition VII, using an appropriate volume of the sample.

VI. Organic Nitrogen [11, 12, 13]

The Kjeldahl method, using copper sulphate as a catalyst and potassium or sodium sulphate to raise the boiling point of the sulphuric acid, is recommended as the standard procedure. However, in most sewages the addition of potassium or sodium sulphate may be eliminated from the digestion procedure.

Direct Nesslerization of the digestate requires less attention, but for strong sewages is much less accurate than distillation into standard acid. The former method is satisfactory for weak sewages, plant control and field work, but distillation into standard acid is preferable where accuracy is desired.

1. *Distillation Into Standard Acid*

Boil or distill off the free ammonia from 100 ml. or more of sample which has been measured into a 500 ml. Kjeldahl flask, and to which has been added 25 ml. of M/2 phosphate buffer solution (see Sec. IV, 4). Add 10 ml. of sulphuric acid (low in nitrogen), 1 ml. of copper sulphate (100 g. per 1000 ml. of water). Digest by boiling in a hood for 20 to 30 minutes after the digestate has become colorless. Cool and add about 250 ml. of ammonia-free water. Make alkaline with sodium hydroxide (500 g. per liter) using the blue copper precipitate as an indicator and distill into 25 ml. or more of 0.05 N acid to which has been added 3 drops of methyl red. Titrate the excess acid with 0.05 N sodium hydroxide.

One ml. of 0.05 N acid is equivalent to 0.7 mg. of nitrogen. A blank should be run on the reagents used and corrections made if necessary.

Calculation:

$$\frac{1000 \times 0.7}{\text{ml. sample used}} \times \text{ml. } 0.05 \text{ N acid} = \text{p.p.m. organic nitrogen.}$$

Nesslerization of the distillate is quite satisfactory on weak sewages.

VII. Total Kjeldahl Nitrogen [11, 12, 13]

Determine total nitrogen as described in VI for organic nitrogen except that the free ammonia is not distilled off before the digestion with acid. Many laboratories prefer to determine the total nitrogen and to obtain the organic nitrogen by subtracting the free ammonia nitrogen as determined by direct Nesslerization, Sec. IV, from the total nitrogen. (The term total nitrogen as used here does not include nitrite and nitrate nitrogen.)

VIII. Nitrite Nitrogen

Determination of the nitrite content shall be made on fresh samples because conversion of nitrite into nitrate and into ammonia by bacterial action proceeds uninterruptedly. (See 1, 3.) Results are to be reported in terms of the nitrogen equivalent.

1. *Reagents*

(*a*) Sulphanilic acid solution. Dissolve 8 g. of purest sulphanilic acid in 1 liter of 5 N acetic acid (sp. gr. 1.041, or 285 ml. glacial acetic acid diluted to 1 liter). This is practically a saturated solution.

(*b*) α-naphthylamine-acetate solution. Dissolve 5 g. of solid α-naphthylamine in 1 liter of 5 N acetic acid. (285 ml. diluted to 1 liter.) Filter the solution through washed absorbent cotton.

(*c*) Sodium nitrite stock solution. Dissolve 1.1 g. of silver nitrite in nitrite-free distilled water; precipitate the silver with sodium chloride solution and dilute to 1 liter.

(*d*) Standard sodium nitrite solution. Dilute 100 ml. of solution (*c*) to 1 liter, then dilute 50 ml. of this solution to 1 liter with sterilized nitrite-free distilled water, add 1 ml. of chloroform, and preserve in a sterilized bottle. One ml. = 0.0005 mg. N = 0.001642 mg. NO_2.

(*e*) Aluminum hydroxide. Electrolyze ammonia-free water, using aluminum electrodes. Wash the precipitate until it is free from chloride, ammonia, and nitrite. Or dissolve 125 g. of potassium or ammonium alum in 1 liter of distilled water. Precipitate the aluminum by cautiously adding ammonium hydroxide. Wash the precipitate in a

large jar by successive additions and decantations of distilled water until free from chloride, nitrite, and ammonia.

(*f*) Fuchsin solution: 0.1 g. of basic fuchsin per liter.

2. *Procedure*

Place in a standard Nessler tube 0.1 to 10 ml. of the sample, decolorized if necessary with nitrite-free aluminum hydroxide and dilute to 50 ml. At the same time prepare in Nessler tubes a set of standards, by diluting to 50 ml. with nitrite-free water, various amounts of standard nitrite. The following volumes of standard solution are suggested: 0.0, 0.1, 0.2, 0.4, 0.7, 1.0, 1.4, 1.7, 2.0, and 2.5 ml. (containing 0.0005 mg. N per ml.). Add 1 ml. of sulphanilic acid solution and 1 ml. of the α-naphthylamine acetate solution to the sample and to each standard. Mix thoroughly and allow to stand 10 minutes; then compare the sample with the standards. Do not allow the sample to stand more than 30 minutes before making the comparison.[14]

Permanent standards may be prepared[15] by matching the nitrite standards with dilutions of the fuchsin solution. Fuchsin standards have been found to be sufficiently accurate for waters high in nitrite and for sewage. The standards should be checked once a month and kept out of the bright sunlight.

Calculation:

$$\frac{\text{ml. standard}}{\text{ml. of sample}} \times 0.5 = \text{p.p.m. nitrite nitrogen.}$$

IX. Nitrate Nitrogen

The following simplified reduction method with direct Nesslerization is recommended as the standard procedure for sewages and effluents. However, because so many still prefer the phenoldisulphonic acid method it is also included as a standard method.

REDUCTION METHOD

1. *Reagents*

(*a*) Sodium or potassium hydroxide. Dissolve 250 g. of the hydroxide in 1.25 liter of distilled water. Add several strips of aluminum foil and allow the evolution of hydrogen to continue over night. Concentrate the solution to 1 liter by boiling.

(*b*) Aluminum foil. Use strips of pure aluminum about 10 cm. long, 6 mm. wide, and 0.33 mm. thick, weighing about 0.5 g.

2. *Procedure*

Add 2 ml. of nitrate-free sodium hydroxide (*a*) to 100 ml. of sample and concentrate by boiling in a casserole, to about 20 ml. to remove free

ammonia. Rinse this concentrated sample into a 50 ml. Nessler tube (or graduated test tube), make up to the mark with ammonia-free water, add one strip of aluminum foil and cover the tube with a loosely fitting stopper (or bunsen valve) to protect the solution from dust. In laboratories where ammonia fumes are present in quantities the ammonia trap described in Standard Methods of Water Analysis, Pt. I, Sec. XI should be used in place of the loosely fitting stopper. Allow the reduction to take place at 20° C. or above for 6 hours, or over night. After reduction, Nesslerize 1 to 5 ml. of the clear reduced liquid after dilution to 50 ml. with ammonia-free water and compare with ammonia standards. If the reduced liquid is not clear, 15 to 20 ml. of it may be clarified in a test tube with a few drops of 10 per cent copper sulphate as in the direct Nesslerization of free ammonia, and from 1 to 5 ml. of the clarified liquid Nesslerized. This latter procedure will prevent cloudy tubes.

Calculation:

1.

$$\frac{\text{ml. } NH_4Cl \text{ standard}}{\text{ml. reduced soln. Nesslerized}} \times 5 = \text{p.p.m. nitrate plus nitrite nitrogen}$$

or

$$\frac{\text{mg. N in standard}}{\text{ml. reduced soln. Nesslerized}} \times 500 = \text{p.p.m. nitrate plus nitrite nitrogen.}$$

2. By subtracting nitrite nitrogen from (1), nitrate nitrogen is obtained.

In strong sewages urea may hydrolize to ammonia and indicate nitrates which are not present. However, on oxidized effluents this error is very remote.

In some industrial wastes and oxidized effluents from trade waste treatment, *very* high nitrite concentration is encountered. The nitrite correction (2) may be eliminated by adjusting the reaction of the sample to pH 7.5, adding 2000 p.p.m. ammonium chloride and evaporating to dryness on a water bath. Wash down with distilled water and evaporate to dryness. Take up the residue in 100 ml. of distilled water, add 5 ml. of 40 per cent nitrate-free sodium hydroxide and boil down to about 25 ml. to expel ammonia. Proceed with the reduction and Nesslerization as described above and report results as nitrate nitrogen, the nitrites having been volatilized on evaporation with the ammonium chloride.[16]

PHENOLDISULPHONIC ACID METHOD

3. *Reagents*

(*a*) Phenoldisulphonic acid. Dissolve 25 g. of pure white phenol in 150 ml. of c.p. concentrated sulphuric acid. Add 75 ml. of fuming

sulphuric acid (15 per cent SO_3), stir well, and heat for two hours at about 100° C.

(*b*) Potassium hydroxide solution. Prepare a 12 N solution by dissolving 673 g. of c.p. KOH in 1 liter of distilled water.

(*c*) Standard nitrate solution. Dissolve 0.7216 g. of recrystalized potassium nitrate in 1 liter of water. Evaporate 50 ml. of the solution to dryness in a water bath. Moisten the residue quickly and thoroughly with 2 ml. of the phenoldisulphonic acid, and rub with a glass rod to insure intimate contact. Dilute to 500 ml. This is the standard solution, 1 ml. of which contains 0.01 mg. of nitrogen, or 0.04427 mg. of NO_3.

(*d*) Standard silver sulphate solution. Dissolve 4.397 g. of silver sulphate, free from nitrate, in 1 liter of distilled water. 1 ml. of this solution is equivalent to 1 mg. of Cl.

4. *Procedure*

Filter 30 to 35 ml. of sewage or effluent through a filter paper and evaporate 25 ml. (or less) of the filtrate to dryness on a water bath. Thoroughly moisten the residue with 1 ml. of phenoldisulphonic acid and dilute to 20 ml. with distilled water. Make the solution alkaline with the potassium hydroxide solution (not over 5 or 6 cc.) until a maximum yellow color is obtained. Filter this solution into a 50 ml. Nessler tube and make the volume up to the mark with distilled water. Make up color standards containing 0.3, 0.5, 0.7, 1.0, 1.5, and 2.0 ml. of standard nitrate solution (*c*), add 2 ml. of the potassium hydroxide solution (*b*) to each standard and make up to the 50 ml. mark with distilled water. Compare the unknown with the color standard and calculate as follows:

$$\frac{\text{ml. of standard}}{\text{ml. of sample used}} \times 10 = \text{p.p.m. nitrate nitrogen.}$$

Notes: Chlorides greater than 120 p.p.m., nitrites greater than 1.0 p.p.m. and carbohydrates interfere with the accuracy of this method. The chlorides may be removed by precipitation with the silver sulphate solution and filtration; the nitrites may be oxidized to nitrates in acid solution with hydrogen peroxide or potassium permanganate and corrected for by subtraction.

X. Oxygen Consumed from Permanganate

The determination of oxygen consumed from permanganate is rapidly losing favor among sanitary chemists for sewage and effluent work, because it does not differentiate between stable and unstable organic materials and, therefore, is not proportionate to the oxygen demand conditions in the samples. It is of value in estimating the strength of certain trade wastes and sewages when the biochemical oxygen demand

(B.O.D.) cannot be determined, or in conjunction with the determination of the B.O.D.

The concentrations of potassium permanganate and sulphuric acid, and the temperature of digestion used in the procedure, control the final oxygen consumed results and, therefore, the technic must be identical with Standard Methods if the results are to be comparable.

Nitrites, ferrous iron, sulphide and other oxidizable substances reduce permanganate, therefore corrections for them should be made. Direct titration of the acidified sample with the permanganate solution in the cold, using a 3-minute period for reaction, serves this purpose quite well.

1. *Reagents*

(*a*) Dilute sulphuric acid. Dilute 1 volume of c.p. sulphuric acid with 3 volumes of distilled water. This solution may be oxidized with permanganate but this is not necessary, because a blank determination on all reagents must be run.

(*b*) Ammonium oxalate solution. Dissolve 0.8880 g. of c.p. ammonium oxalate in 1 liter of distilled water. 1 ml. is equivalent to 0.1 mg. of oxygen. This solution *is not stable* and should be prepared each month.

(*c*) Standard potassium permanganate. Dissolve 0.4 g. of c.p. potassium permanganate in 1 liter of distilled water. Add 10 ml. of this solution and 10 ml. of the dilute sulphuric acid to 100 ml. of distilled water in an Erlenmeyer flask, and digest 30 minutes in boiling water. Add 10 ml. of the standard ammonium oxalate solution, and then titrate to a pink coloration with the standard potassium permanganate. This destroys the oxidizing capacity of the distilled water. Now add 10 ml. of the oxalate solution and titrate with the potassium permanganate solution. Adjust the permanganate solution so that 1 ml. is equivalent to 1 ml. of the oxalate solution or to 0.1 mg. of available oxygen. Preserve this solution in a dark glass bottle.

2. *Procedure*

In an Erlenmeyer flask, dilute 1 to 10 ml. of sewage or 25 to 50 ml. of an oxidized effluent to 100 ml. with distilled water. Add 10 ml. of the potassium permanganate solution and 10 ml. of the dilute sulphuric acid. Digest for 30 minutes in a boiling bath. Add 10 ml. of the oxalate solution and titrate back with the standard potassium permanganate solution to a faint pink color. Run a blank determination on the distilled water and reagents used. Subtract the ml. of permanganate solu-

tion used in the blank from that used on the sample and calculate as follows:

$$\frac{\text{ml. KMnO}}{\text{ml. sample}} \times 100 = \text{p.p.m. oxygen consumed.}$$

Note: Some chemists prefer to use a standard permanganate solution of 10 times the above strength with strong sewages. This usually gives much higher results because of the stronger oxidizing capacity of the stronger permanganate. These higher results are comparable in magnitude with the B.O.D. of the crude sewage. However, on oxidized effluents the stronger permanganate oxidizes more of the stable organic matter and gives results many times greater than the B.O.D. On some wastes and digested liquors the results will be lower, rather than higher.

XI. Dissolved Oxygen

The Winkler process [17] for the determination of dissolved oxygen depends on the formation of a precipitate of manganous hydroxide in a glass-stoppered bottle completely filled with the water under examination. The oxygen dissolved in the water is rapidly absorbed by the manganous hydroxide, forming a mixture of higher oxides which, on acidification in the presence of an iodide, releases iodine in a quantity chemically equivalent to the oxygen content of the sample. The liberated iodine is then titrated with a standard solution of sodium thiosulphate. Various modifications of this general procedure are available for counteracting the effect of interfering substances.

SAMPLING FOR DISSOLVED OXYGEN

Sample bottles of the type commonly used in oxygen demand determinations (240 to 380 ml.) are very satisfactory. At the start of an investigation it may nevertheless be desirable, for convenience and uniformity, to select a 250-ml. glass-stoppered bottle (8-ounce tincture).

In general, the sampling arrangements should be such as to insure at least a three-fold displacement of the liquid in the sampling bottle without entrainment of air bubbles.[23] In sampling from streams, ponds or tanks of moderate depth, this may be accomplished by the use of apparatus of the type shown in Figure 1.

It will be noted that the sample bottles begin to fill as soon as the above-mentioned sampling devices are submerged. If a sample is required from a considerable depth, it may be desirable to use a sampler provided with a valve release.

SELECTION OF A METHOD OF PROCEDURE

Huge errors may be introduced in dissolved oxygen work (and also in oxygen demand work) by the neglect of proper precautions in the

presence of interfering substances such as nitrites, ferrous salts, organic matter, sulphites, etc., or by an improper application of the various modifications of the Winkler method which are designed to overcome this type of interference.

I. *The regular Winkler* procedure should be used in determining the dissolved oxygen content of pure waters.

II. *The Rideal-Stewart* [18] or *permanganate modification* of the Winkler method should be used when appreciable amounts of nitrites [19] or

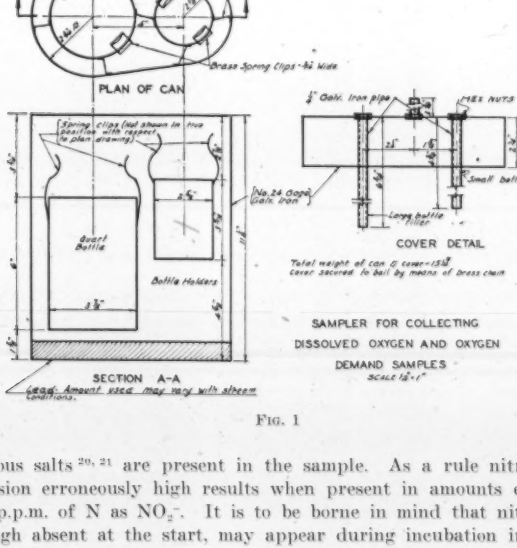
Strips extend from top of bottle holders to bottom of can.
Brass Spring Clips - ½ Wide
PLAN OF CAN
Spring clips (not shown in true position with respect to plan drawing)
Quart Bottle
Bottle Holders
SECTION A-A
Load: Amount used may vary with stream conditions.
¾" Galv. Iron pipe
1 1/8" NUTS
No. 24 Gage Galv. Iron
Small bottle filler
Long bottle filler
COVER DETAIL
Total weight of can & cover - 15 lb.
Cover secured to bail by means of brass chain
SAMPLER FOR COLLECTING
DISSOLVED OXYGEN AND OXYGEN
DEMAND SAMPLES

FIG. 1

ferrous salts [20, 21] are present in the sample. As a rule nitrites will occasion erroneously high results when present in amounts exceeding 0.1 p.p.m. of N as NO_2^-. It is to be borne in mind that nitrites, although absent at the start, may appear during incubation in oxygen demand tests. Interference due to nitrites may also be expected in dealing with effluents from biological oxidation devices.

High results may be obtained in the presence of ferric salts owing to the liberation of iodine from iodides in the final step of the Winkler process, especially if the titration is delayed.[20, 21] On the other hand, one part per million of ferrous iron will occasion an apparent loss of about 0.14 p.p.m. of dissolved oxygen.

The permanganate modification is not recommended as a corrective for interference due to relatively stable forms of organic matter such as

sugars, starches, etc., which are not readily acted upon by permanganate in the cold. A preliminary treatment with permanganate of samples containing such substances is unprofitable and may actually be detrimental. The permanganate modification also fails in the presence of sulphite wastes.

III. *The alkaline-hypochlorite modification*[22] should be used when the sample contains sulphites, thiosulphates, polythionates, free chlorine or hypochlorites. (Paper mill wastes, chlorinated effluents, etc.)

THE STANDARDIZATION OF THIOSULPHATE SOLUTIONS

The following reagents and solutions are required:

1. *Starch Solution.*—About 0.5 per cent. Grind 5 or 6 grams of arrowroot starch in a mortar with a small quantity of water. Pour into 1 liter of boiling water, stir and allow to settle overnight. Use the clear supernatant. This solution may be preserved with (1) 1.25 g. salicylic acid per liter or (2) 4 g. zinc chloride and 10 g. of potassium iodide per liter.

2. *Dilute Acid for Standardization.*—1.8 *M* H_2SO_4 approximately. (One volume of concentrated sulphuric acid poured into 9 volumes of water.)

3. *Potassium or Sodium Iodide Crystals.*—Free from iodates.

4. *Stock Sodium Thiosulphate Solution.*—24.82 grams of $Na_2S_2O_3 \cdot 5H_2O$ per liter, corresponding to a 0.1 *M* solution. The solution may be preserved: (1) by the addition of ammonium carbonate (1.5 grams per liter) and chloroform (5 ml. per liter *after* making up to the mark), or (2) by the addition of 1 *M* NaOH at the rate of 4 ml. per liter.

5. *Standard Sodium Thiosulphate Solution.*—0.025 *M*. Prepared by diluting 250 ml. of stock thiosulphate solution (0.1 *M*) to 1 liter or by weighing out 6.205 grams of $Na_2S_2O_3 \cdot 5H_2O$ and making up to 1 liter with freshly boiled distilled water.

The keeping qualities of the 0.025 *M* thiosulphate solution are somewhat improved by the addition of 5 ml. of chloroform after making up to the mark. The chloroform also serves to keep the burettes clean. This solution, however, will deteriorate quite rapidly and should be made up fresh every few weeks. The strength of the solution may be checked with a standard solution of biniodate or bichromate.

6. *Potassium Biniodate Solutions.*—A stock solution of strength equivalent to that of the 0.1 *M* thiosulphate contains 3.250 grams of $KIO_3 \cdot HIO_3$ per liter in accordance with the following reaction:

$$2KIO_3 \cdot HIO_3 + 20KI + 11H_2SO_4 = 11K_2SO_4 + 12H_2O + 12I_2$$

A biniodate solution equivalent to the 0.025 *M* thiosulphate solution contains 0.8124 gram of $KIO_3 \cdot HIO_3$ per liter and may be prepared by

diluting 250 ml. of the stock biniodate solution to 1 liter.

Potassium bichromate may be substituted for potassium biniodate if difficulty is experienced in securing the latter reagent. A solution of potassium bichromate equivalent to the 0.1 *M* sodium thiosulphate should contain 4.903 grams of $K_2Cr_2O_7$ per liter.

For the standardization about 5 grams of potassium iodide (free from iodate) are dissolved in a 250-ml. Erlenmeyer flask with a small quantity of distilled water and 10 ml. of dilute sulphuric acid (1 : 10) are added, followed by 40.00 ml. of standard biniodate (or bichromate) solution. The liberated iodine is then titrated with the thiosulphate solution, adding starch toward the end of the titration when a pale straw color is reached. Exactly 40.00 ml. of thiosulphate should be required when the solutions under comparison are of equal strength.

REAGENTS FOR THE WINKLER METHOD

The following solutions are required when the unmodified Winkler procedure is used:

1. *Manganous Sulphate Solution.*—480 grams of $MnSO_4 \cdot 4H_2O$ or 400 grams of $MnSO_4 \cdot 2H_2O$ dissolved in distilled water, filtered and made up to 1 liter. When uncertainty exists regarding the water of crystallization, a solution of equivalent strength may be obtained by adjusting the specific gravity of the solution to a value of 1.270 at 20° C. The manganous sulphate solution should liberate only traces of iodine when added to an acidified solution of potassium iodide. (Absence of manganic and ferric salts.)

2. *Alkaline-Iodide Reagent.*—500 grams of NaOH (or 700 grams of KOH) and 135 grams of NaI (or 150 grams of KI) per liter. Potassium and sodium salts may be used interchangeably. The reagent should be practically free from carbonates. It should not give any color with starch when diluted and acidified. (Absence of nitrites, iodates and ferric salts.)

3. *Concentrated Sulphuric Acid.*—Specific gravity, 1.83–1.84. The strength of this acid is about 18 *M* (36 *N*). Hence 1 ml. is equivalent to about 3 ml. of the alkaline iodide solution.

REAGENTS FOR THE PERMANGANATE (RIDEAL-STEWART) MODIFICATION

The following solutions should be prepared when the permanganate modification of the Winkler method is to be used as a preliminary treatment for the destruction of nitrites:

1. *Potassium Permanganate Solution.*—6.32 grams of $KMnO_4$ per liter.

2. *Oxalate Solution.*—2 grams of potassium oxalate $(COOK)_2 \cdot H_2O$, in 100 ml. of distilled water. One ml. of this oxalate solution will suffice

for the neutralization of about 1.1 ml. of the permanganate solution. Under certain conditions this solution deteriorates rapidly. An oxalic acid solution of equivalent strength (1.4 grams of $(COOH)_2 \cdot 2H_2O$ in 100 ml. of distilled water) may be used.

In the presence of appreciable amounts of iron salts, the following solutions will also be required:

1. *Fluoride Solution.*—40 grams of $KF \cdot 2H_2O$ dissolved in distilled water and made up to 100 ml.

2. *Phosphoric Acid.*—85 per cent H_3PO_4.

REAGENTS FOR THE ALKALINE-HYPOCHLORITE MODIFICATION

The following solutions are required when the alkaline-hypochlorite modification is applied for the oxidation of sulphites, thiosulphates, etc.:

1. *Alkaline-Hypochlorite Solution.*—1 *M* NaOCl and 0.1 *M* NaOH. This solution may be prepared by passing chlorine gas through a 2.1 *M* NaOH solution, with cooling, until a 1 ml. test portion of the chlorinated solution requires about 20 ml. of 0.1 *M* thiosulphate for the neutralization of the iodine released upon acidification in the presence of an iodide.

The pharmaceutical preparation "Hyclorite," if fresh, is approximately molar in respect to hypochlorite. On adding 1 volume of 1 *M* NaOH to 9 volumes of "Hyclorite," a precipitate of $Ca(OH)_2$ is formed, which settles out readily. The supernatant liquor is in reasonable conformity with the above specifications.

The alkaline-hypochlorite solution should be kept tightly stoppered when not in use and its strength should be checked at least every week.

2. *Iodide Solution.*—1 *M* KI or NaI. Prepared by dissolving 17 grams of KI or 15 grams of NaI in distilled water and making up to 100 ml. This solution should be preserved by the addition of 1 ml. of 1 *M* NaOH to each 100 ml. of solution.

3. *Sodium Sulphite Solution.*—0.05 *M* (0.1 *N*). Contains 6.3 grams Na_2SO_3 per liter.

THE WINKLER METHOD

The following procedure is applicable to the determination of dissolved oxygen in the *absence* of unstable forms of organic matter and of such readily oxidizable mineral substances as nitrites, ferrous salts, polythionates, etc.

The volume of Winkler reagents to be added is based on the assumption that bottles of the size recommended herein (250 ml.) are being used. With larger bottles (300 to 380 ml.), 2 ml. each of the Winkler reagents should be used. With smaller bottles (125 ml. or less), it will be sufficient to add 0.5 ml. each or the three Winkler reagents (manganous sulphate solution; alkaline-iodide solution; concentrated sulphuric acid).

Procedure

1. *Add 1 ml. of the manganous sulphate followed by 1 ml. of alkaline-iodide solutions.* These reagents should be added well below the surface of the liquid so as to avoid loss of precipitate on replacing the stopper.

2. *Shake well, by inverting the bottle several times, so as to distribute the precipitate uniformly throughout the bottle.*

As the precipitate settles readily, the dissolved oxygen in the upper part of a bottle will not be completely absorbed unless the period of shaking is suitably prolonged. If uncertainty exists on this point, or else if a clear supernatant is not obtained, the shaking should be repeated. With sea water a 10-minute period of contact with the manganous hydroxide may be required.

Numerous forms of organic matter are capable of direct oxidation by dissolved oxygen at pH values of 12 or thereabouts (corresponding to the degree of alkalinity obtained when alkaline-iodide is added to a sample). For this reason the test should be carried through on individual samples when interference by organic matter is suspected and the period of alkalinization should be reduced to the minimum time consistent with the complete absorption of dissolved oxygen.

When 1 ml. of manganous sulphate solution is used with 250 ml. of sample it may safely be assumed that the absorption of dissolved oxygen will be practically complete if the precipitated manganous hydroxide is agitated *continuously* for 40 to 50 seconds. With 2 ml. of manganous sulphate, a shorter period of shaking (20 to 25 seconds) is sufficient. If the sample is then acidified as soon as the precipitate has settled, accurate results will be obtained in the presence of organic matter in amounts corresponding to 1000 parts per million of either dextrose or peptone.

Larger amounts of organic materials may be present (say 5000 p.p.m. or 0.5 per cent of dextrose) without introducing an appreciable error, if the sample is acidified before the precipitate has settled (that is, immediately after shaking), suitable allowance being made for the slight loss of precipitate by displacement on adding the acid. This procedure should be followed with sulphite wastes.

3. *Acidify with 1 ml. of concentrated sulphuric acid, allowing the acid to run down the neck of the bottle.*

4. *To avoid the reduction of manganic salts by organic materials,* it is desirable that the sample be shaken as soon as possible after the addition of acid when organic substances are present.

The liberated iodine diffuses slowly. It should be uniformly distributed throughout the bottle before withdrawing an aliquot part of the sample for titration.

5. *In the absence of ferric salts* and certain forms of organic matter (acetic acid, for example), it may be possible to delay the titration for several days without introducing an appreciable error.

The titration may be delayed for one hour, even in the presence of 100 p.p.m. of ferric iron, if potassium fluoride has been added in the preliminary treatment. In the absence of fluoride, the titration should be performed *immediately* after the final acidification when the ferric iron content exceeds, say 10 p.p.m.

The titration may be delayed for several days, even in the presence of 100 p.p.m. of iron, if phosphoric acid (4 ml. of 85 per cent H_3PO_4) is substituted for sulphuric acid in the final acidification.

6. *A correction for the loss of dissolved oxygen* by displacement with the reagents should be applied to the amount measured out for titration. Thus, when 1 ml. each of the manganous sulphate and alkaline-iodide reagents are added to a 250-ml. bottle, the amount measured out for titration should be:

$$200 \times \frac{250}{250 - 2} = 201.6 \text{ ml.}$$

if it is desired to titrate 200 ml. of the original sample.

When the preliminary treatment with permanganate has been used with 250-ml. bottles,

$$200 \times \frac{250}{250 - 5.7} = 204.7 \text{ ml.}$$

should be measured out if it is desired to titrate 200 ml. of the original sample.

7. *Titrate with 0.025 M thiosulphate* until the amount of iodine remaining is equivalent to about 0.5 ml. of thiosulphate solution, corresponding to a pale straw color in the sample.

8. *Add 1 or 2 ml. of freshly prepared starch solution* and titrated rapidly to the first disappearance of the blue color. Subsequent recolorations owing to the presence of traces of iron salts or to the catalytic effect of nitrites, etc., should be disregarded.

9. *When several samples are being run at the same time*, four or five aliquot portions may be poured out at one time if the titrations are not delayed, provided that iron salts and nitrites are not present in more than traces.

The loss of iodine by diffusion into the air is appreciable if the oxygen content exceeds, say 7 p.p.m. and the titration is unduly delayed.

10. *When the volume titrated* is equivalent to 200 ml. of the original sample, the dissolved oxygen content in parts per million is numerically

the same as the required volume of 0.025 *M* thiosulphate expressed in milliliters. In general,

$$\text{P.p.m of dissolved oxygen} = \frac{200 \text{ (ml. of 0.025 } M \text{ thiosulphate)}}{\text{(ml. of sample titrated)}}$$

11. *Example:* If 100 ml. of sample require 4.50 ml. of 0.025 *M* thiosulphate, the dissolved oxygen content is

$$\frac{200}{100} \times 4.50 = 2 \times 4.50 = 9.00 \text{ p.p.m.}$$

THE PERMANGANATE MODIFICATION

The permanganate (Rideal-Stewart) modification of the Winkler method should be used in determining the dissolved oxygen content of partly nitrified effluents and, in general, whenever the sample contains 0.1 p.p.m. or more of nitrogen as nitrite. Through the use of fluorides and of phosphoric acid the applicability of the permanganate modification may also be extended to mine wastes and other samples containing large amounts (100 p.p.m. or more) of iron.

Procedure

All reagents, excepting the sulphuric acid, should be added well below the surface of the liquid:

1. *Add 0.7 ml. of concentrated sulphuric acid.* It is important that the amount called for should not be exceeded. The 0.7 ml. portion, for that reason, is best added with a 2-ml. pipette.

2. *Add enough of permanganate solution to obtain a violet tinge* which persists after shaking (usually 1 ml.).

With samples whose behavior is fairly well known, the proper amount of permanganate may be added all at once.

With samples of unknown or variable composition, trial amounts (1 ml. or less) should be added with shaking and a suitable delay allowed between additions. When an additional amount of permanganate appears necessary, it should be added while a faint violet tinge still persists (that is, before the appearance of brownish colorations due to intermediate oxides).

To avoid too great a dilution of the sample, a stronger solution of permanganate should be used when large amounts (5 ml. or more) of this reagent are required.

3. *In practice, a period of standing of five minutes* at ordinary temperatures after obtaining a permanent violet coloration is sufficient to reduce the nitrite content to a value which will not cause undue interference.

4. *Potassium oxalate solution is added to destroy the excess* of per-

manganate whenever the oxidation of the interfering substances (readily oxidizable materials) is judged complete.

As a trial amount which should not be greatly exceeded, it is recommended that 0.5 ml. of the 2 per cent oxalate solution be first added. If after standing for five minutes the color of permanganate still persists, an additional 0.5 ml. portion should be added. It is important that perfect decolorization be secured, as even a trace of brownish coloration will react with iodides on acidification, giving results which may be entirely too high.

In the presence of iron salts, decolorization is delayed to a surprising extent (24 hours under certain conditions), even when the correct amount of oxalate has been added. In order to hasten the decolorization when iron salts are known to be present, potassium fluoride (2 ml. of 40 per cent solution) should be added along with the permanganate or at any other stage of the preliminary treatment.

For the reason that ferric salts in the presence of oxalates are readily reduced to the ferrous condition by light (very rapidly so in direct sunlight), the decolorization must proceed in the dark when iron salts are present in amounts greater than 1 p.p.m.

5. *When the permanganate is fully decolorized*, add 1 ml. of manganous sulphate solution followed by 3 ml. of alkaline-iodide reagent. Do not add less than the indicated amounts.

The foregoing steps constitute the preliminary treatment of a sample for the destruction of nitrites, sulphides, etc., and for the oxidation of ferrous salts to the ferric condition. The subsequent treatment is the same as when the Winkler method is used, starting with the shaking of the sample after the addition of the Winkler reagents. (See 2, under "Procedure," for the Winkler method.) It should not be assumed that organic materials are absent when the permanganate treatment has been used.

THE ALKALINE-HYPOCHLORITE MODIFICATION

Polythionates and similar compounds in paper mill wastes break down readily in alkaline solution to form sulphites and thiosulphates. The oxidation of such compounds must, therefore, be carried to the formation of sulphates before the addition of the alkaline-iodide solution. A preliminary treatment, according to the Rideal-Stewart modification, is ineffective as the oxidation to sulphates of sulphites, thiosulphates and polythionates is incomplete when permanganate is used. The error with samples containing 0.25 per cent of digester waste from the manufacture of sulphite pulp may amount to 7 or 8 parts per million of dissolved oxygen, the results appearing too low. With such samples a practical degree of accuracy may be secured through preliminary treatment with hypochlorite and iodine, as described below. At best, how-

ever, this procedure gives low results, the error with 0.25 per cent of digester waste amounting to about 1 part per million. Delays at any stage of the process should be avoided.

The procedure given below consists in a preliminary treatment of the sample with an alkaline solution of sodium hypochlorite for the decomposition of polythionates and for the oxidation of sulphites, thiosulphates, etc., to sulphates. The subsequent treatment (see "3" below, under "Procedure") calls for the removal of the undecomposed hypochlorite through the liberation of iodine which is in turn removed with a dilute solution of sodium sulphite. Dissolved oxygen may then be determined by the regular Winkler procedure.

RESIDUAL CHLORINE

Beginning with "3" the procedure given below should also be used when dissolved oxygen is to be determined in samples containing free chlorine or hypochlorites. For the reason that tetrathionates are readily decomposed in alkaline solution, sodium thiosulphate should not be used for the removal of iodine liberated from iodides by free chlorine and its compounds.

A preliminary treatment with hypochlorite is effective in counteracting interference due to nitrites or ferrous salts although, for this purpose, it is perhaps simpler to use the permanganate treatment.

Procedure

1. *Add enough of the alkaline-hypochlorite reagent* (1 *M* NaOCl and 0.1 *M* NaOH) to oxidize the sample, avoiding a great excess.

With river water polluted with sulphite waste, 0.2 ml. of the alkaline-hypochlorite reagent may be added as a trial amount. In oxygen demand work, add 1 ml. of hypochlorite solution for each ml. of digester waste present in the bottle under examination.

2. *Mix well by inverting rapidly a few times.* The duration of this treatment should not greatly exceed 20 seconds.

If the period of contact with the alkaline-hypochlorite is unduly prolonged, low results will be obtained, owing to the absorption of dissolved oxygen by the decomposition products of the organic matter.

3. *Acidify the sample by adding 1 ml. of 1.8 M* H_2SO_4 (10 per cent solution) and add 1 ml. of *M* KI (or NaI) to release iodine. Shake.

4. *Neutralize the liberated iodine with 0.05 M sodium sulphite*, using 0.2 ml. of starch solution as an inside indicator. If the end point is overstepped, the blue color should be restored with 0.1 ml. portions of potassium biniodate solution of equivalent strength.

The reaction between sulphites and iodine to form sulphates is complete only in very dilute solution. Under the given experimental condi-

tions the results will be relatively low when more than 3 ml. of 0.05 *M* (0.1 *N*) sulphite are required for the neutralization of the iodine. On the other hand, if only 0.1 ml. or thereabouts of 0.05 *M* (0.1 *N*) sulphite are required, it may be assumed that an insufficient amount of alkaline-hypochlorite has been used.

5. *Add 1 ml. of manganous sulphate solution and 1.3 ml. of alkaline-iodide solution.*

A slight excess of the alkaline-iodide reagent is used in order to neutralize the acid added during the preliminary treatment.

For the subsequent treatment see 2, under "The Winkler Method," beginning with the shaking of the sample.

SCHEMATIC OUTLINE FOR DISSOLVED OXYGEN DETERMINATION

For convenience of reference, the following outline has been prepared by condensing the detailed instructions given above:

The Winkler Method

1. When a preliminary treatment is unnecessary the test is started by adding 1 ml. each of the manganous sulphate and alkaline-iodide solutions. Lesser amounts may be added under certain conditions.

When a preliminary treatment with permanganate has been applied, add 1 ml. of manganous sulphate and 3 ml. of alkaline-iodide reagent.

When the alkaline-hypochlorite modification has been used, add 1 ml. of manganous sulphate and 1.3 ml. of alkaline-iodide.

2. In the practical absence of organic matter it is permissible to shake each bottle vigorously, passing from one bottle to another of a series. A second shaking is advisable. In the presence of much organic matter, treat each sample individually as directed in detailed instructions.

3. Acidify with concentrated sulphuric acid.

4. Distribute the liberated iodine uniformly throughout the bottle.

The Titration

5. In general, the titrations should be commenced within one hour after the final acidification. (See detailed instructions for tolerance in special cases.)

6. For the titration it is convenient to pour out a portion of the iodine solution which corresponds to 100 or 200 ml. of the origianl sample.

7. Titrate to a pale straw color with 0.025 *M* thiosulphate.

8. Add 1 or 2 ml. of starch solution and titrate rapidly to the first disappearance of the blue color, neglecting subsequent recolorations.

9. The titration should not be delayed, once the sample has been poured out.

Computations

10. In general,

$$\text{P.p.m of dissolved oxygen} = \frac{200 \text{ (ml. of 0.025 } M \text{ thiosulphate)}}{\text{(ml. of sample titrated)}}$$

The results may be converted into ml. oxygen per liter by multiplying the p.p.m. by 0.698, or into per cent saturation by using the solubility tables in Standard Methods of Water Analysis or other hand books.

Preliminary Treatment with Permanganate and Fluoride

1. Add 0.7 ml. of concentrated sulphuric acid.
2. Add enough of permanganate solution to obtain a violet tinge.

Add 2 ml. of fluoride solution if appreciable amounts of iron salts are present.

3. Allow to stand until the reaction of the permanganate with readily oxidizable substances is judged complete (5 minutes or less).
4. Decolorize with the minimum possible amount of oxalate solution. The decolorization must proceed in the dark when iron salts are present.
5. The subsequent treatment of the sample is the same as when the regular Winkler method is used.

Preliminary Treatment with Alkaline-Hypochlorite

1. Add a suitable amount of alkaline-hypochlorite reagent.
2. Shake for 20 seconds.
3. Add 1 ml. each of 1.8 M H_2SO_4 (10 per cent solution) and 1 M KI. Shake.
4. Add 0.2 ml. of starch solution and neutralize the iodine with 0.05 M (0.1 N) sulphite solution, restoring the blue color with 0.1 ml. portions of biniodate solution of equivalent strength.
5. The subsequent treatment of the sample is the same as when the regular Winkler method is used.

XII. Biochemical Oxygen Demand

The biochemical oxygen demand (frequently referred to as B.O.D.) of sewage, sewage effluents, polluted waters or industrial wastes is the oxygen in parts per million required during stabilization of the decomposable organic matter by aerobic bacterial action. Complete stabilization requires more than 100 days at 20° C., but such long periods of incubation are impracticable in any but research investigations, consequently a much shorter period of incubation is used. Incubation for 1, 2, 5, 10 or 20 days at 20° C. is customary, and the 5-day period is recommended as the standard procedure. Conversion of the data from one incubation period to another, or from one temperature to another may

be made.[37, 38] It is essential that normal growth of bacteria and plankton be permitted in the diluted samples, consequently reliable results cannot be obtained if the diluted sample contains caustic alkalinity, acid, free chlorine, or any other bactericidal substance. In such cases the bactericidal substance must be neutralized, reduced or removed if possible, and the sample seeded with normal sewage organisms. Recent work [31] indicates that the presence of protozoa or other higher microorganisms are necessary for a completely satisfactory flora.

In order to obtain constant results from two or more dilutions, the dilution water must be free from an appreciable oxygen demand of its own, and also must contain no germicidal properties due to residual chlorine, caustic alkalinity or copper from the lining of the still. A satisfactory water should have a depletion of less than 0.2 p.p.m. of dissolved oxygen in 5 days incubation at 20° C., and should not be bactericidal. These requirements have led to the use of synthetic dilution waters, composed of *pure* distilled water and sodium bicarbonate or phosphate buffer solutions. Distilled water alone and chemically treated tap waters should not be used. Chloramine treated tap waters should be passed through charcoal filters before distillation, to remove the chloramine compounds which are volatile.

1. *Reagents and Apparatus*

(a) Standard Dilution Waters

1. Sodium bicarbonate dilution water. A good quality distilled water (containing less than 0.05 p.p.m. copper) is thoroughly aerated in five-gallon carboys and stored until ready for use. When ready for use add 0.3 g. per liter of c.p. sodium bicarbonate and thoroughly mix, but do not aerate. The pH value of this freshly prepared water should be less than 8.0. If this solution is to be used over a period of a few days or week it should be kept tightly stoppered to prevent the loss of carbon-dioxide and a subsequent rise in the pH. If possible this dilution water should be kept in the 20° C. incubator.

2. Phosphate dilution water. Stock Buffer Solution. Dissolve 34 g. of c.p. potassium acid phosphate (KH_2PO_4) in about 500 ml. of distilled water. Add 1 *M* sodium hydroxide (40 g. NaOH per liter) until a pH of 7.2 is reached. About 175 ml. of the 1 *M* NaOH will be required. The mixture is then diluted to 1 liter.[30] The phosphate dilution water is then prepared by adding 1.25 ml. of the stock buffer solution to each liter of aerated distilled water (free of copper salts, etc.). This produces a water containing about 50 p.p.m. of mineral salts.

3. A similar water may be prepared by adding 1 ml. on the M/2 phosphate buffer solution (Sec. IV, Ammonia Distillation) to each liter

of the aerated distilled water. This produces a water containing about 105 p.p.m. of mineral salts.

The most generally used water is the sodium bicarbonate water. The U. S. Public Health Service laboratories at Cincinnati have made an extensive study of these waters and reports as follows:[30] "The 5-day oxygen demand results are not greatly affected when a bicarbonate solution containing 300 p.p.m. of $NaHCO_3$ is used instead of a phosphate solution buffered at 7.2. Over shorter periods of incubation the phosphate dilution water appears to give somewhat higher results (absence of lag). Conversely, when periods of incubation are prolonged beyond five days, the bicarbonate water gives the highest results owing, possibly, to the favorable effect of relatively high pH values on the activity of nitrifying organisms."

(*b*) Incubation bottles should be of 250 to 300 ml. capacity, with ground-glass stoppers, cleansed with chromic acid mixture, carefully rinsed and dried before use.

(*c*) As a precaution against drawing air into the dilution bottle during incubation it is recommended that a water seal be used.

(*d*) An air incubator or water bath thermostatically controlled to within 1° of 20° C. should be used.

2. *Procedure*

1. If the sample contains caustic alkalinity or acid, neutralize it to brom thymol blue (about pH 7.0) with dilute hydrochloric acid or sodium carbonate and seed with sewage organsims.

2. Where necessary seeding is to be accomplished by adding stale sewage (about 1 ml. per liter) or river water (10 to 20 ml. per liter) to the dilution water. In the case of chlorinated sewages or effluents the seeding may not be effective unless all the chlorine and chloramine compounds have been dissipated. (See tentative procedure, page 473.)

3. Supersaturation. Samples collected during the winter months or from localities where algae are actively growing may be supersaturated (over 9.17 p.p.m.) with dissolved oxygen with reference to a temperature of 20° C. To allow for fluctuations in temperature, the following procedures should be applied whenever the dissolved oxygen content exceeds, say, 9.0 p.p.m. The procedures are given in order of preference and in each case the duration of the treatment is to be determined empirically.

(*a*) Suction may be applied to a bottle partly filled with the sample after the temperature of the sample has been raised to approximately 20° C. The release of oxygen is greatly accelerated by agitation.

(*b*) The sample may be mildly preheated in a water-bath. The excess oxygen is then liberated by agitation and the sample is cooled to

20° C. The temperature of the water-bath should not exceed, say, 30° C.

(*c*) Compressed air may be passed through the sample. The possibility of diluting the sample with deaerated water should also be borne in mind.

4. Prepare several dilutions of the prepared sample in order to cover the range of depletions expected (usually 0.1 to 1.0 per cent of strong trade wastes; 1 to 5 per cent of normal and settled sewages; 5 to 25 per cent of oxidized effluents; and 25 to 100 per cent on polluted river waters). This may be accomplished as follows: Carefully siphon a 500 or 1000 ml. graduated cylinder half full of dilution water without entrainment of air. Add the quantity of carefully mixed (not aerated by vigorous shaking) sample to make the desired dilution and fill to the 500 or 1000 ml. mark with dilution water. Mix well with a plunger type mixing rod, avoiding the entrainment of air. Siphon the mixed dilution into incubation bottles (one for incubation and one for determining the dissolved oxygen in the mixture, if desired), tightly stopper and fill the water seal. Prepare succeeding dilutions of less concentration in the above manner or by adding dilution water to the unused portion of the preceding dilution. These dilutions may be made in graduated flasks, thus attaining greater accuracy in measurement, but less accuracy in good mixing technic.

a. The dilution technic is greatly simplified when suitable amounts of sample are added directly to bottles of known capacity, the bottles having been previously filled half full of dilution water. Then fill the bottle into the neck with just sufficient water so that the stopper can be inserted without leaving an air bubble, stopper and fill the seal. This procedure presupposes that the volumes of the dilution bottles have been accurately determined. Dilutions greater than 1 to 100 (1 per cent) should be made according to 4.

5. Fill two bottles completely with dilution water, stopper and fill the seal of one for incubation. The other bottle is for determining the dissolved oxygen of the dilution water before incubation, as a check on the quality of the dilution water. The 5-day oxygen consumption should not exceed 0.2 p.p.m.

6. Determine the dissolved oxygen on the undiluted sample. In practice this determination on crude sewages and settled effluents is not usually necessary, but it should be made on oxidized effluents.

7. Incubate the blank and the diluted samples for 5 days at 20° C., and then determine the dissolved oxygen of the incubated samples and the blank, using the permanganate modification of the Winkler method, unless experience has shown that nitrites in the incubated samples will be less than 0.1 p.p.m. (in special cases some of the other modifications

may be necessary). These dilutions showing 40 to 70 per cent depletion of the initial oxygen content shall be considered most reliable.

Calculation:

Let p = per cent of dilution water expressed as a decimal,
P = per cent of sample expressed as a decimal,
D.O. = p.p.m. oxygen determined by titration,
$D.O._1$ = (D.O. in incubated blank × p) + (D.O. in sample × p) = calculated D.O. available in dilution for depletion by sample,
$D.O._2$ = D.O. in diluted sample after incubation.
Then $(D.O._1 - D.O._2) \times 1/P$ = B.O.D. in p.p.m.

Note: No correction is made for the small loss (or gain) of dissolved oxygen in the dilution water during incubation because the calculation is based on the incubated blank and not on the unincubated dilution water.

If it is desired to determine the immediate oxygen demand of the sample the D.O. of the diluted sample should be run on an additional diluted sample within 15 minutes after making the dilution and this quantity subtracted from the calculated initial D.O. of the dilution. This initial D.O. is calculated as in (1) above except that the initial D.O. of the dilution water before incubation is used in place of that of the incubated blank.

Calculation:

($D.O._1$ calculated — D.O. of dilution after 15 minutes) × 1/P = immediate oxygen demand in p.p.m.

Tentative Procedure for B.O.D. on Chlorinated Sewage [36]

1. If not alkaline to phenolphthalein.

(*a*) To a portion of a sample of 100 ml. to 500 ml., depending on the chlorine residual present, add 10 ml. of 20 per cent potassium iodide (KI) solution and titrate the liberated iodine with N/40 sodium acid sulphite ($NaHSO_3$) solution which has been standardized against N/10 iodine solution immediately before using. (Test for nitrite before making this determination because of possibility of false chlorine test).

(*b*) To a suitable volume, for example 1 liter of sample, add just sufficient N/40 $NaHSO_3$ solution to de-chlorinate, as calculated from the above titration. After mixing, test 10 ml. with ortho-tolidine solution to check absence of free chlorine. To another 10 ml. portion add 1 drop of N/100 iodine solution and a few drops of starch solution; a blue color should develop, indicating no appreciable excess of $NaHSO_3$.

(*c*) Proceed with the B.O.D. dilutions, determining the initial D.O. of the dechlorinated sample and seeding the dilution water with unchlorinated sewage. (See B.O.D. procedure).

2. If alkaline to phenolphthalein.

(*a*) To 100 ml. of sample add a few drops of phenolphthalein indicator and titrate with N/50 HCl to complete discharge of the pink color.

(*b*) To another portion of sample 100 ml. to 500 ml., depending on the chlorine residual present, add the volume of N/50 HCl required in (*a*), then add 10 ml. of 20 per cent potassium iodide solution and titrate with N/40 $NaHSO_3$ solution.

(*c*) To a suitable volume, for example 1 liter of sample, add N/50 HCl and N/40 $NaHSO_3$ as calculated from the above titrations, mix, test for free chlorine and for excess sodium acid sulphite as in 1-(b) and proceed with the B.O.D. dilutions as in 1-(c).

In the final B.O.D. calculation, correction is made for the addition of sodium acid sulphite solution and hydrochloric acid solution if added.

3. For small residuals (less than 0.5 p.p.m.).

To a suitable volume of sample, for example 1 liter, add N/40 $NaHSO_3$ as calculated from the chlorine residual (determined by orthotolidine test, noting the absence of nitrite by making the usual Griess-Ilosvay Test). Thus if residual chlorine is 0.33 p.p.m. add to 1 liter 0.37 ml. of N/40 $NaHSO_3$. After 10 minutes test 10 ml. with orthotolidine to note absence of chlorine. It may also be expected that residuals of a few tenths parts per million will often disappear on standing for an hour or two, making the addition of $NaHSO_3$ unnecessary.

NOTES

1. Sodium acid sulphite is used in preference to sodium thiosulphate since with the latter, on oxidation with chlorine, tetrathionate is formed and in the D.O. determination, in the presence of alkali, tetrathionate is converted into sulphite and thiosulphate, which reduce manganic hydroxide to the *ous* form. Further, the reduction of chlorine by sodium sulphite is affected less by the hydrogen ion concentration than is the case with sodium thiosulphate.

2. N/40 sodium acid sulphite solution loses strength on standing and its strength must be checked frequently; however, it is much more stable than sodium sulphite, Na_2SO_3.

3. Phenolphthalein indicator is not destroyed by chlorine in moderate concentration.

4. Incidentally the $NaHSO_3$ titration provides a means for the accurate determination of the residual chlorine present, especially for large residuals.

5. Chloramines are also reduced by sodium sulphite.

XIII. Relative Stability of Effluents

Relative stability may be defined as the per cent ratio of oxygen available as dissolved, nitrite and nitrate oxygen to the total oxygen required to satisfy the biochemical oxygen demand, Pt. I, Sec. XII. This percentage or ratio may be approximately indicated by determining the number of days required to exhaust the available oxygen in the sample, using methylene blue as an indicator.

The use of this determination is rapidly decreasing, as it is being replaced by the more exact determinations of dissolved oxygen, nitrite, nitrate and biochemical oxygen demand. However, for small plants with limited laboratory facilities the stability towards methylene blue fulfills a very useful service in indicating the satisfactory operation of the biological oxidation processes.

Effluents which contain caustic alkalinity or acidity should be neutralized to brom thymol blue and seeded with sewage bacteria before adding the indicator. If bactericidal substances are present that cannot be removed, the method is inapplicable.

The theoretical relation between the time required for decolorization of methylene blue at 20° C. and the relative stability percentage is given in Table 13 and is represented by the relation $S = 100(1 - 0.789^t)$ in which S is the stability in per cent and t is the time in days required for decolorization at 20° C.

1. *Reagent*

(*a*) Methylene blue. Dissolve 0.5 g. of U.S.P. quality methylene blue in distilled water and make up to 1 liter.

2. *Procedure*

Fill a 150 ml. glass stoppered bottle with sample, avoiding aeration. Add exactly 0.4 ml. of methylene blue indicator solution below the surface of the liquid. Incubate at 20° C. with a water seal, observing the samples daily until decolorization takes place. Report the days required for decolorization, or if preferred, the relative stability percentage shown in Table 13. If other sized incubation bottles are used a proportional amount of the indicator solution must be used.

TABLE 13.—Relative Stability Numbers

Time Required for Decolorization at 20° C.	Relative Stability *S*	Time Required for Decolorization at 20° C.	Relative Stability *S*
Days	*Percentage*	*Days*	*Percentage*
0.5	11	8.0	84
1.0	21	9.0	87
1.5	30	10.0	90
2.0	37	11.0	92
2.5	44	12.0	94
3.0	50	13.0	95
4.0	60	14.0	96
5.0	68	16.0	97
6.0	75	18.0	98
7.0	80	20.0	99

XIV. Total Solids

Evaporate 100 ml. of sample in an ignited and tared evaporating dish, dry to constant weight at 103° C., cool in desiccator and weigh. (This usually requires 2½ hrs. drying at 103° C.)

$$\text{mg. solids} \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. total solids.}$$

Total volatile and fixed solids are determined by igniting the above total solids at 600° C. in an electric muffle (usually requiring 10 to 15 minutes).

$$\text{mg. loss} \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. volatile solids}$$

$$\text{mg. residue} \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. fixed solids.}$$

XV. Suspended Solids

1. *Reagent*

(*a*) Asbestos cream. Prepare a cream with distilled water of acid-washed, medium fiber asbestos, which is prepared particularly for Gooch crucible determinations.

2. *Procedure*

Prepare a mat of asbestos fiber, from 2 to 3 mm. thick, in a Gooch crucible by gentle suction. Wash with 100 ml. of distilled water, dry at 103° C., cool and weigh. If volatile matter is to be determined by ignition the crucible and mat must be *ignited*, cooled and weighed.

Filter from 50 to 100 ml. of sample through the weighed Gooch crucible using suction. Dry at 103° C. for 2½ hours, cool in a desiccator and weigh. (The volume of sample should yield less than 30 mg. of solids).

$$\text{mg. solids} \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. suspended solids.}$$

Ignite at 600° C. in electric muffle (10 to 15 minutes), cool in desiccator and reweigh.

$$\text{mg. loss} \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. volatile susp. solids.}$$

$$\text{mg. residue} \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. fixed susp. solids.}$$

Dissolved solids may be obtained by difference or by determination in a filtered sample, according to above procedures.

The volatilization of organic matter, carbonates and ammonium salts from sewage solids is subject to a great many errors. It should be done in an electric muffle furnace at 600° C. However, in plant control laboratories, ignition may be accomplished over a Meeker burner, heating the dish to a dull red heat until no black carbon specks remain. The ash should be white or reddish brown.

XVI. Settleable Solids

1. *By Volume*

Fill an Imhoff cone to the liter mark with a thoroughly mixed sample. Settle for 1.75 hours, gently stir the sides of cone with a rod or by spinning, settle 0.25 hour longer and record the ml. of settleable solids in the cone. In hot weather it may be necessary to reduce the settling to 1 hour because of floating sludge. Some operators prefer to use the theoretical detention period of the plant settling units.

2. *By Weight*

If it is desired to determine the settleable solids by weight most of the supernatant liquor is siphoned off and the settled solids washed into a tared evaporating dish, evaporated, dried and weighed. The weight in mg. equals p.p.m. settleable solids. Corrections should be made for the dissolved solids in the supernatant liquor which is dried with the settleable solids, if greater accuracy is desired.

XVII. Determination of Grease in Sewage

Grease in sewage includes fats, waxes, free fatty acids, calcium and magnesium soaps, mineral oils, and other non-fat materials. Of the three common fat solvents, petroleum ether, ethyl ether, and chloroform, the latter extracts the greatest amount of grease, and petroleum ether the smallest. All of these solvents extract additional material after the sample has been acidified. This is particularly true of petroleum ether, and to a much smaller extent also true of chloroform. More data are needed regarding the chemical composition of the material extracted by the different solvents, and until this is available it seems best to recommend petroleum ether, ethyl ether and chloroform. In reporting results the extracting agent should always be stated. On the insufficient data available, chloroform appears to be the most satisfactory solvent. However, it has not received sufficiently widespread use to warrant discarding the ether solvents at this time.

Procedure

Concentrate 500 to 1000 ml. of sewage to about 50 ml. in an evaporating dish, scrape all solid matter from the sides of the dish into the liquid

with a rubber tipped glass rod, add a few drops of methyl orange, and make slightly acid with dilute hydrochloric acid. Evaporate to dryness on a water bath, and dry 30 minutes at 100–105° C. Scrape all material on the sides to the center of the dish, with a spatula, and transfer to an extraction thimble. The removal of dry solids is greatly facilitated if cotton or asbestos (preferably the latter) is placed in the dish at the start of evaporation. The amount should equal about 15 ml. in volume. Remove any traces of solids in the dish with pieces of filter paper moistened with the extracting agent, and add this to the contents of the thimble. Place the thimble in a Soxhlet or other extraction apparatus, and extract with petroleum ether, ethyl ether or chloroform for 12 to 16 hours. Evaporate the solvent, dry the flask for 1 hour at 100–105° C., and weigh. The increase in weight gives the weight of grease in the sample.

3. Calculation:

$$\text{mg. grease} \times \frac{1000}{\text{ml. of sample}} = \text{p.p.m. grease}$$

XVIII. Acidity

1. *Reagents*

(*a*) Phenolphthalein indicator. Dissolve 0.5 g. of phenolphthalein in 1 liter of 50 per cent alcohol. Neutralize the solution with 0.02 N sodium hydroxide. The alcohol should be made by dilution with boiled distilled water.

(*b*) Sodium hydroxide: 0.02 N solution.

2. *Procedure*

Titrate 50 ml. of the settled sample in a 50 ml. graduated cylinder, using a plunger type stirring rod, 4 drops of phenolphthalein indicator, and 0.02 N sodium hydroxide solution until the first permanent pink color appears. Express the total acidity as p.p.m. $CaCO_3$ or as milli-equivalents of acid (0.001 N.).

Calculation:

ml. 0.02 N sodium hydroxide $\times$ 20 = p.p.m. acidity as $CaCO_3$

ml. 0.02 N sodium hydroxide $\times$ 0.4 = milli-equivalents of acid (M.E.)

XIX. Alkalinity

1. *Reagents*

(*a*) Sulfuric acid: 0.02 N solution.

(*b*) Phenolphthalein indicator. Sec. XVIII above.

(c) Methyl Orange indicator: Dissolve 0.5 g. of a good grade of methyl orange in 1 liter of distilled water. Keep the solution in the dark.

2. *Procedure*

Titrate 50 ml. of the settled sample in an Erlenmeyer flask with 4 drops of phenolphthalein indicator and 0.02 N sulphuric acid solution until the pink color (if there is any) disappears. If no pink color is produced when the indicator is added, hydroxide and normal carbonate are absent and there is no alkalinity to phenolphthalein. The number of ml. of acid used times 20 equals the p.p.m. alkalinity to phenolphthalein as $CaCO_3$ (P).

Add 2 drops of methyl orange indicator to this solution, and if the color is yellow, titrate with 0.02 N sulphuric acid until the faintest pink coloration appears. The total number of ml. of acid used in the phenolphthalein and methyl orange titrations times 20 equals the p.p.m. of total alkalinity as $CaCO_3$ (T).

Relations Between Alkalinity to Phenolphthalein and That to Methyl Orange in Presence of Hydroxide, Carbonate, and Bicarbonate

Result of titration.*	Value of radicle expressed in terms of calcium carbonate.		
	Hydroxide	Carbonate	Bicarbonate
$P=0$	0	0	T
$P<\frac{1}{2}T$	0	$2P$	$T-2P$
$P=\frac{1}{2}T$	0	$2P$	0
$P>\frac{1}{2}T$	$2P-T$	$2(T-P)$	0
$P=T$	T	0	0

* T = Total alkalinity in presence of methyl orange or indicator of similar range.
P = Alkalinity in presence of phenolphthalein or indicator of similar range.

The alkalinity may be expressed as milli-equivalents by using the factor 0.4 instead of 20 in the above calculations for P and T, or by dividing the alkalinity as $CaCO_3$ by 50.

XX. Hydrogen Ion Concentration

Colorimetric methods using indicators and buffer standards are most satisfactory for control and research problems on sewage and most trade wastes except in a few cases where strongly colored wastes interfere. Many of the commercial indicator sets on the market are quite satisfactory, but with each set the so-called permanent color standards should be frequently checked against standard buffer solutions, and the indicator solutions must always be purchased from the maker of the particular set in use. Procedures are furnished with the sets and usually

consist in adding 0.25 to 1 ml. of indicator solution to 10 ml. of settled sample and comparing the color with standards in a compensating colorimeter. (See Pt. II, Sec. V, 3.)

XXI. Chlorides

1. *Reagents*

(*a*) Standard sodium chloride solution. Dissolve 16.4858 g. of pure fused sodium chloride in distilled water and make up to 1 liter. Dilute 100 ml. of this stock solution to 1 liter: 1 ml. contains 1 mg. of chloride radicle.

(*b*) Standard silver nitrate solution. Dissolve about 2.40 g. of silver nitrate crystals in 1 liter of distilled water. Titrate (see 3) against the standard salt solution, correct for the error due to variations in the volume of the liquid by means of the formula[2] $X = 0.003V + 0.02$, in which X = the correction in ml. to be deducted from the volume of silver nitrate solution used, and V = ml. of liquid at the end of the titration. Adjust the solution so that 1 ml. will be exactly equivalent to 0.5 mg. of chloride radicle.

(*c*) Potassium chromate indicator. Dissolve 50 g. of neutral potassium chromate in a little distilled water. Add silver nitrate to produce a slight red precipitate; after a day or two, filter and dilute the filtrate to 1 liter with distilled water.

(*d*) Aluminum hydroxide. As directed in X, 1 (*e*).

2. *Procedure*

If the sample contains hydrogen sulphide, acidify 50 ml. of sample with sulphuric acid and oxidize the sulphides by heating with hydrogen peroxide for a few minutes. Cool and neutralize with sodium bicarbonate.

If sulphide is absent omit the above step, neutralize the sample to phenolphthalein if it is acid or alkaline. Add 1 ml. of the potassium chromate indicator and titrate against a white surface with the standard silver nitrate solution until a faint permanent reddish coloration is perceptible. If more than 7 or 8 ml. of silver nitrate are required use a smaller sample, diluted to 50 ml. with distilled water.

Make corrections for the error due to variations in volume as directed in 1 (*b*). The correction to be deduced is 0.2 ml. unless unusual accuracy is required. The following calculation includes the correction.

Calculation:

$$\frac{(\text{ml. of } AgNO_3 - 0.2) \times 500}{\text{ml. of sample}} = \text{p.p.m. chloride.}$$

XXII. Residual Chlorine

1. *Colorimetric Determination*

The high alkalinity of sewage requires an ortho-tolidine reagent with greater acidity than that for water analysis and therefore the following proceedure is recommended. The stronger acid also greatly reduces the interference by nitrites.

(*a*) Ortho-tolidine solution. Dissolve 1 g. of ortho-tolidine in 1 liter of hydrochloric acid (180 ml. HCl sp. g. 1.18 to 1.19 diluted to 1 liter). It is sometimes easier to dissolve the ortho-tolidine by grinding in the 180 ml. of strong acid and then diluting to 1 liter with distilled water.

(*b*) Standard chlorine solution. Dilute 2.5 ml. of B–K solution (a pharmaceutical preparation of approximately 3.5 per cent sodium hypochlorite) to 1 liter with distilled water. This will give a solution of approximately 0.1 mg. of chlorine in 1 ml. of solution. This solution is not permanent and must be made up often and standardized frequently. Dilute solution so that 1 ml. contains 0.1 mg. of chlorine. (H.T.H. or Penchlor may also be used to make up this solution.)

Standardization. To 150 ml. of distilled water add 1–2 g. of potassium iodide crystals and dissolve, add 50 ml. of chlorine solution, 1 ml. of concentarted hydrochloric acid and allow 5 minutes for liberation of iodine. Then titrate with 0.025 N sodium thiosulphate, using starch solution in the usual way, until the blue color has disappeared. 1 ml. of 0.025 N sodium thiosulphate is equivalent to 0.886 mg. of chlorine, and

$$\text{mg. of } Cl_2 \text{ per ml.} = \frac{\text{ml. 0.025 N sodium thiosulphate} \times 0.886}{50}$$

To a 10 ml. sample of chlorinated sewage that has had at least a ten minute contact period (and has been warmed to 20° C. if the sample was cold) add 1 ml. of ortho-tolidine solution. Allow 5 minutes for development of maximum color and compare with color standards, made in the same manner, containing 0., 0.2, 0.4, 0.6, 0.8, and 1.0 mg. of chlorine per liter in a turbidity compensating colorimeter. One ml. of the standard chlorine solution diluted to 100 ml. with distilled water is equivalent to 1 ml. per liter (1 p.p.m.) of chlorine. The standards are not permanent and must be made up with each determination. Satisfactory permanent standards, including a colorimeter, are on the market.

2. *Titration*

This method is not accurate in the presence of nitrites, because nitrites in the presence of acid liberate iodine from potassium iodide.

Procedure:

To 200 ml. of settled sewage in an Erlenmeyer flask, add about 1 g. of potassium iodide and 1 ml. of concentrated hydrochloric acid, shake

well and allow 5 minutes in the dark for the liberation of iodine. Titrate the liberated iodine with *freshly prepared* 0.001 N sodium thiosulphate (40 ml. 0.025 N sodium thiosulphate diluted to 1 liter with freshly boiled distilled water) using starch solution as the indicator.

Calculation:

ml. 0.001 N sodium thiosulphate $\times$ .1777 = p.p.m. residual chlorine

XXIII. Chlorine Demand

The chlorine demand of sewage or effluents is different when chlorinated by sodium hypochlorite solution and chlorine water, and since chlorine water is used most generally for sewage chlorination it is better to determine the chlorine demand with chlorine water.[41]

1. *Reagents*

(*a*) Chlorine water. By passing chlorine gas through distilled water a solution is made which contains slightly more than 1 g. of chlorine per liter (1 ml. equals approximately 1 mg. Cl_2). This solution is not stable and must be standardized each time it is used, by the procedure recommended in Sec. XXII, using 5 ml. of the chlorine water instead of 50 ml. as recommended.

2. *Calculation*

$$\text{mg. of } Cl_2 \text{ per ml.} = \frac{\text{ml. 0.025 N sodium thiosulphate} \times 0.886}{5}$$

(b) Ortho-tolidine solution (Sec. XXII).

Definition.[41] The chlorine demand (C.D.) of a sewage shall be the parts per million of chlorine required to be added as chlorine water (1 g. Cl_2 per liter) to 250 ml. of sewage to produce a trace (0.1 p.p.m.) of residual chlorine after 10 minutes contact of chlorine and sewage. The residual chlorine is to be determined with ortho-tolidine solution according to the procedure in Sec. XXII, 1.

3. *Preliminary Procedure*

To a 250 ml. sample add the standard chlorine water 0.5 ml. at a time (equivalent to 2.0 p.p.m. of chlorine) with gentle stirring until a spot plate test using 1 drop of ortho-tolidine solution and 0.25 ml. of the sample gives a yellow color. Call this the immediate chlorine demand. (I.C.D.)

4. *Final Procedure*

To a 250 ml. sample add the total amount of chlorine water determined as immediate chlorine demand plus from 0.25 to 1.0 ml. in addition. Gently stir and in exactly 10 minutes test a portion for

residual chlorine with ortho-tolidine according to the procedure in Sec. XXII, 1. Repeat this procedure until the amount of chlorine is determined which conforms to the definition for chlorine demand.

Calculation, when using 250 ml. of sample:

(ml. of Cl_2 soln.) $\times$ (mg. of Cl_2 per ml.) $\times 4 =$ p.p.m. chlorine demand (C.D.)

XXIV. Sulphides (Total, Fixed and Volatile)

Titration of sulphide in sewage with standard iodine solution is not satisfactory because other substances in the sewage react with iodine. For control and many research problems the following colorimetric method is satisfactory.[42]

1. *Reagents*

1. Antimony potassium tartrate solution. Dissolve 50 g. of the *c. p.* salt in distilled water and make up to 1 liter.

2. Hydrochloric acid (1:1).

3. NaCl crystals.

4. Standard sodium sulphide. Dissolve 7.5 g. of fresh C.P. Na_2S9H_2O in distilled water and make up to 1 liter. 1 ml. of this solution is equivalent to 1 mg. of S. This should be made up frequently and standardized with 0.025 N iodine and 0.025 N sodium thiosulphate using starch as an indicator. 1 ml. of 0.025 N thiosulphate is equivalent to 0.4005 mg. sulphur as S.

Permanent color standards that are satisfactory have not been developed.

2. *Procedure*

Total sulphides.

Add 1 ml. of antimony solution and a few crystals of NaCl to a 4 oz. clear glass bottle and add 100 ml. of sample to be tested. A yellow color indicates free hydrogen sulphide. Mix and add 1 ml. of hydrochloric acid solution (1:1). Prepare color standards in a similar manner by adding 0.05, 0.1, 0.2, 0.3, 0.4 and 0.5 ml. of standard sodium sulphide solution to 100 ml. distilled water, thus preparing a series equivalent to 0.5, 1.0, 2.0, 3.0, 4.0, and 5.0 p.p.m. sulphide as S. Compare the colors, compensating for turbidity in the usual manner and report the results in p.p.m. total sulphides, as elemental sulphur (S).

If the sample contains more than 5 p.p.m. S an appropriate dilution of the sample should be made with sulphur-free distilled water.

Volatile and fixed sulphides have been determined by thoroughly aerating the sample for a few minutes until no odor of H_2S is observed; then repeating the above procedure for total sulphides. If all the H_2S has been removed by aeration no color should develop until the acid is

added. R. C. Pomeroy questions[43] the advisability of attempting this separation because of the effect of pH on the equilibrium constant. The volatile and fixed division of the sulphur should be considered as qualitative, not quantitative.

Titration Method

The sulphides may be determined by volatilization with CO_2 gas, absorption in zinc acetate solution, and titration with standard iodine and thiosulphate.[44]

1. *Procedure*

Introduce 500 ml. of sample in an aeration cylinder of 1 liter capacity which is equipped with an alundum filter disc in the bottom for introducing the CO_2 gas, and an outlet tube at the top for connection with a 10 bulb absorption tube, the latter containing 2 per cent zinc acetate solution for absorbing the hydrogen sulphide evolved. Pass CO_2 through the sample for 3 or 4 hours to expel the free hydrogen sulphide. To the zinc acetate H_2S solution in the absorption train add 10–20 ml. of approximately 0.025 N iodine, mix, add 5 ml. concentrated HCl, stopper and shake. Transfer liquid to a beaker and back-titrate the excess iodine with 0.025 N sodium thiosulphate, using starch solution as the indicator. Run a blank titration on the same number of ml. of standard iodine and reagents added above. 1 ml. 0.025 N sodium thiosulphate is equivalent to 0.4008 mg. of hydrogen sulphide as sulphur.

Calculation:

$$(\text{ml. thiosulphate for blank} - \text{ml. thiosulphate for sample}) \times 0.4 \times \frac{1000}{\text{ml. sample}} = \text{p.p.m. volatile hydrogen sulphide as S}$$

For total sulphides acidify (2 — 5 ml. concentrated H_2SO_4) 500 ml. of sample in the aeration cylinder and proceed as above. Only one-half to one hour aeration with CO_2 is necessary. Considerably more than 20 ml. of 0.025 N I_2 may be necessary if the total sulphides are high.

Calculation of total sulphide is the same as for volatile hydrogen sulphide. The fixed sulphide is determined by difference.

Again, the analytical separation of volatile and fixed sulphur is of doubtful value from a quantitative point of view.[43] Volatile sulphur depends on the final pH of the solution and the length of time the volatilization process is continued. However, it may have a certain practical value in relation to odors.

PART II. EXAMINATION OF SEWAGE SLUDGE AND MUDS

1. Collection of Samples

Great care must be taken in sampling sewage sludges, sludge banks, muds, etc. No definite procedure can be given, but every possible pre-

caution should be taken to get a representative sample, whether it is obtained from a tank, a sludge pile or from the bottom of a river.

When analysis cannot be made immediately, the sample should be stored on ice. The samples cannot be preserved with chloroform if fats are to be determined. Mud samples may be preserved with 5 g. of sodium benzoate, or 2 ml. of concentrated sulphuric acid to each 80 g. of mud.[45]

I. PHYSICAL TESTS

Color, odor, and physical appearance shall be described as best suits the individual case, no standard method being recommended. Temperature shall be recorded in degrees Centigrade.

II. SPECIFIC GRAVITY

Weigh a wide-mouthed flask or bottle to the nearest 0.1 g., empty, filled completely with distilled water, and again filled completely with the sample. The specific gravity is equal to the ratio of weight of the sludge or mud to that of the same volume of water. Record the specific gravity to the third decimal place.

In case the mud does not flow readily, add as much of it to the bottle as possible, without exerting pressure, and weigh. Fill bottle with water and weigh again, making sure all entrained air bubbles have escaped.

Calculation:

$$\frac{\text{Wt. of mud}}{\text{Wt. of water to fill bottle} - \text{wt. of water added to mud}} = \text{specific gravity of the mud}$$

III. SETTLEABILITY OF ACTIVATED SLUDGE

Allow 1000 ml. of activated sludge to settle in a 1000 ml. graduated cylinder. Record the volume occupied by the sludge at various intervals of time (10, 20, 30, 45, 60, etc., minutes) plot these volumes against the time and obtain a curve indicating the rate of settling. For plant control a 30-minute settling period is rather generally used.

IV. SLUDGE INDEX. (S.I.) [46, 47]

The sludge index is the volume in ml. occupied by 1 g. of activated sludge after settling the aerated liquor for 30 minutes. A one-liter sample is collected at the outlet of the aeration tanks, settled 30 minutes in a 1000 ml. graduated cylinder and the volume occupied by the sludge reported in per cent or ml. The sample is thoroughly mixed, or the original sample taken, and the suspended solids determined and reported in per cent by weight or p.p.m.

Computation:

$$1.\ \frac{\text{Settling } \%}{\text{Susp. solids } \%} = \text{S.I.}$$

$$2.\ \frac{\text{ml. settleable sludge}}{\text{p.p.m. suspended matter}} \times 1000 = \text{S.I.}$$

V. REACTION (ACIDITY, ALKALINITY AND pH)

Determine the acidity, alkalinity and pH value of the liquor that separates from the sludge on standing. Both acidity and alkalinity titrations are made using a tall cylinder and plunger as described for titrating carbon dioxide (Pt. I, Sec. XVI).

1. *Acidity*

Measure into a 90–100 cc. cylinder (hydrometer jar) 50 ml. of supernatant liquor from the sample. Care must be taken to minimize the loss of CO_2 in thes operations. Add 3 drops of phenolphthalein indicator solution and titrate with 0.05 N sodium hydroxide solution to the first permanent pink color. A plunger type of stirring rod is used.

Calculation:

(1) ml. 0.05 N hydroxide $\times$ 50 = p.p.m. acidity to phenolphthalein expressed as $CaCO_3$

(2) ml. 0.05 N hydroxide $\times$ 1 = acidity in milli-equivalents

2. *Alkalinity*

Measure 50 ml. of the supernatant liquor (see acidity above) in a glass cylinder. Add 3 drops of phenolphthalein and titrate with 0.05 N sulphuric acid until pink color has disappeared. Record ml. 0.05 N acid used as P. Add 3 drops of methyl orange and continue titration until the solution is a faint pink. Record the total ml. of 0.05 N acid used, including that for the phenolphthalein titration, as T.

Calculation: When 50 ml. of supernatant sludge liquor is used

$P \times 50$ = p.p.m. alkalinity to phenolphthalein as $CaCO_3$
$T \times 50$ = p.p.m. alkalinity to methyl orange as $CaCO_3$

To express these results in milli-equivalents of alkalinity, use the factor 1 instead of 50 in the above calculations.

3. *Hydrogen Ion Concentration (pH)* [48, 49]

The pH of the liquor in normal sewage sludge, particularly digested sludge, is dependent to a considerable extent on the carbon dioxide equilibrium. Any technic which allows for loss of carbon dioxide from the liquid will give pH results that are too high. The simpler the technic, the less shaking, filtering, aeration and agitation, the more accu-

rate the results will be. The colorimetric determination of pH, using indicators and buffer standards, is the most satisfactory. Many satisfactory commercial sets with more or less permanent standards are on the market. The permanent standards should frequently be checked against standard buffers, and the indicator solutions must be purchased from the maker of the particular set used, because the color intensity of different indicator salts varies considerably, and such salts must be matched with the color intensity of the permanent standards.

The sludge sample should be collected with as little aeration as possible. If sludge liquor will separate from the sludge in 30 minutes standing the pH should be run on this liquor. However, the buffer capacity of most sludge liquors is sufficient so that the sludge may be diluted 1 to 5 with aerated distilled water (pH 6.8–7.0), settled and the pH determined on the supernatant liquor. This latter procedure is not advised if liquor will separate without dilution. Filtration of the sludge is not recommended because of the possible loss of CO_2.

The procedure with most colorimetric sets consists in superimposing 10 ml. of the sample in front of the color standards, to compensate for color and turbidity, and comparing a 10 ml. sample to which has been added the requisite quantity of indicator solution (usually 0.25 to 1.0 ml.).

VI. MOISTURE AND SOLIDS

1. *Moisture and Total Solids*

Weigh 50 g. of sludge as rapidly as possible into a tared evaporating dish. Evaporate to dryness over night in a 103° C. oven, cool in a desiccator and reweigh.

Calculation:

$$\frac{\text{Loss of weight} \times 100}{\text{wt. of wet sluge}} = \text{per cent moisture}$$

$$\frac{\text{Wt. of residue} \times 100}{\text{wt. of wet sluge}} = \text{per cent total solids}$$

2. Volatile Solids. (Includes volatile inorganic solids.)

Ignite the residue from the determination of moisture in an electric muffle at 600° C. (dull red heat) for 60 minutes. Cool in a desiccator and reweigh.

Calculation:

$$\frac{\text{Loss of weight} \times 100}{\text{wt. of residue}} = \text{per cent volatile solids}$$

The determination of both total and volatile solids is subject to considerable error due to the loss of ammonium carbonate while drying in

(1) making the total solids and the volatile solids lower than they should be.

VII. TOTAL NITROGEN (IN THE DRY SLUDGE)

Procedure:

Dry a sample of sludge or mud in a 103° C. drying oven, grind thoroughly to a fine powder and dry again for 30 minutes at 103° C.

Weigh accurately 1.0 g. of dried sludge or 1.0 to 5.0 g. of dried mud into a 500 ml. Kjeldahl flask, add 20 ml. of sulphuric acid, 1 ml. of 10 per cent copper sulphate solution and 10 g. of potassium or sodium sulphate, mix thoroughly, digest slowly until frothing ceases and then for 30 minutes after the liquor becomes colorless. Cool, dilute to 300 ml. with ammonia-free water, and proceed as directed in Pt. I, Sec. VI, except that 0.5 N instead of 0.05 N sulphuric acid is used, and 0.5 N sodium hydroxide is used for back titration. 1 ml. of 0.5 N acid is equivalent to 0.007 g. of nitrogen.

Calculation:

$$\frac{\text{ml. 0.5 N acid consumed} \times .7}{\text{g. of sample}} = \text{per cent nitrogen as N}$$

The above method as it stands is far from being the total nitrogen of the wet sludge. At best it is Kjeldahl nitrogen minus ammonium bicarbonate or carbonate lost on drying. A more rational method for determining the total nitrogen in wet sludge is as follows:

1. *Ammonia Nitrogen*

Rapidly weigh approximately 20 grams of wet sludge (5 per cent solids) to — 0.1 mg. in a weighing bottle or crucible. Wash sample into a 500 ml. Kjeldahl flask with distilled water or ammonia-free water, dilute to 250 ml. and proceed as directed in Part I, Sec. IV, distillation procedure except that 100 ml. of distillate is collected in standard H_2SO_4 and the excess acid titrated to methyl red end point. A piece of paraffin added to the flask will prevent frothing. Calculate in p.p.m. N.

Organic Nitrogen

To the residue from ammonia determination add 1 ml. 10 per cent $CuSO_4$ solution, 5–10 g. Na_2SO_4 or K_2SO_4 and 20 ml. of H_2SO_4, digest and proceed as directed Pt. II, Sec. VII. Calculate in per cent N of the total solids, determined on a separate sample.

VIII. GREASE (FATS)[49]

Of the three common grease solvents, petroleum ether, ethyl ether and chloroform, the first extracts the least and the latter the most grease. All these solvents extract additional material after the sample has been

acidified. This is particularly true of the petroleum and ethyl ethers and to a much smaller extent with chloroform. Until more data are obtained, it seems advisable to recommend all three extractives. (See Pt. I, Sec. XVII.)

Procedure

1. *Direct Extraction*

Weigh approximately 5 g. of dry sludge into an extraction thimble, and extract with petroleum ether, ethyl ether or chloroform in a Soxhlet or other continuous extractor for 12–16 hours. Boil off the excess solvent, dry the residual grease at 100–105° C., cool in a desiccator, and weigh. Repeat drying and weighing to constant weight.

2. *Acidification and Extraction*

Place approximately 5 g. of dry sludge in a porcelain dish, add water and methyl orange, and make distinctly acid with hydrochloric acid. Evaporate to dryness on a steam bath, and dry 30 minutes at 100–105°. Transfer the solid quantitatively to an extraction thimble, extract, dry and weigh as directed in 1. Report results as per cent grease in the dried sludge or mud, stating what solvent was used.

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Editor's Note: This report is being issued as a separate reprint to facilitate study, trial and comment on the methods of analysis recommended by the committee.

It is urged that criticisms be sent to Dr. W. D. Hatfield, Decatur Sanitary District, Decatur, Ill., that they may receive consideration for future editions of Standard Methods of Sewage Analysis.

Copies of the reprint can be obtained from W. W. Buffum, Business Manager, 654 Madison Ave., New York City at 50 cents apiece, postage paid.

SEWAGE PLANT LABORATORIES

By Wellington Donaldson *

With Fuller and McClintock, Engineers, New York City

Laboratory measurements are the sound basis of successful sewage treatment, as in all other departments of applied science or industrial achievement. Without such measurements, "sewage" is only a broad generic term for the liquid wastes of a community; "sludge" is a very indefinite material; and any statements as to plant performance, efficiency, or quantities handled are necessarily vague, unsatisfactory and difficult of comparison with other localities.

It is true that the solid and organic contributions to sewage from a large number of human beings are very much the same in amount and kind for all civilized populations with similar habits. The "per capita" human contribution is therefore a widely used and useful expression for estimating the loading of sewage treatment works. However, considerable judgment must be exercised in making use of per capita assumptions to take account of the trade wastes, street wash or other additives to the human contribution. It is also important to remember that treatment works deal with material which may be removed from sewage with widely varying liquid volumes, the amount and range of which it is highly important to take into account if the major treatment structures are to function dependably and efficiently. For these reasons adequate analytical data and flow measurements are highly desirable in advance of design, both of initial works and subsequent additions.

One of the most hopeful auguries for the future science of sewage treatment is the fact that, with the modern type of treatment works now built or projected, an increasing number of young technically-trained men are being attracted to this field of municipal sanitation, more competent studies are being made of sewage treatment problems in various parts of the country and it is becoming common practice to install well equipped laboratories in connection with the new plants. The effects of these trends are already being definitely felt in betterment of operation, improvement of existing processes, development of new and better processes, reduced costs of treatment and more definite information as to performance.

The laboratory facilities provided at American sewage treatment works vary over extremely wide limits as to equipment, scope and cost. At one end of the range is the small plant where a table or window bench

* Presented at the Seventh Annual Meeting of the New York State Sewage Works Association, New York City, January 15, 1935.

in one corner of the building is used at odd moments during the day by the factotum to make a few simple control tests, such as settleable solids, relative stability, residual chlorine and pH. At the other end of the range is the well appointed laboratory of the large municipality or sanitary district which may occupy a separate building and furnish employment to a staff of laboratory workers, including specialists in research. The purpose of this paper is to discuss some of the features of design and equipment of sewage plant laboratories suitable for average communities, of say 10,000 to 50,000, where only one laboratory worker is employed and perhaps part of his time is allotted to other duties.

In considering the needs of what we may call the average plant laboratory, it is well to keep in mind certain requirements not inherent in other laboratories engaged on sewage problems, whether collegiate, purely research or commercial. These requirements may affect materially the location, layout and equipment. The sewage plant laboratory is concerned primarily with handling a number of samples daily on a routine basis according to a definite prescribed schedule and in the most expeditious manner. Upon these routine tests rest the records of plant performance and the quantities reported. A secondary function, although a highly important one, is the conduction of special studies of individual processes with a view toward betterments. A study of the effect of the plant effluent on the receiving waters is often a profitable and necessary function of the plant laboratory. In addition to these more or less perfunctory duties, it may be assumed that the ambitious laboratory worker will undertake certain research problems related to the local situation. The facilities for which, above mere routine requirements, should reasonably be provided.

The cost of equipping and maintaining a plant laboratory is quite modest compared with the total investment and operating cost of the treatment works. In view of the advantages to be expected from improved operation and definite knowledge as to plant performance, the laboratory investment is well justified. Table 1, containing data obtained by questionnaire from 17 plants selected somewhat at random as representative of modern works, indicates that the investment in laboratory, exclusive of building space, ranges from about 0.2 per cent to 2.0 per cent of the total plant investment, usually less than 1 per cent. The failure to include in Table I some well known laboratories, particularly in the west and southwest, is regretted and is due to time limitations for collection of information. Some of these laboratories, as at Chicago, Milwaukee, Ft. Worth, San Antonio, Phoenix, Pasadena and Los Angeles have rendered conspicuous service.

It should be understood that the figures given in Table I refer generally to well equipped laboratories, some of which have been in opera-

TABLE I.—*Size and Cost of Typical Sewage Plant Laboratories*

Works	Pop. Served	Floor Space Sq. Ft.	Bench Space Sq. Ft.	No. Workers	Value of Laboratory				% Plant Invest.	Information By
					Furniture	Apparatus	Chemicals	Total		
Decatur, Ill. (S. D.)	58,000	400	91	1	$1134	$1922[1]	—	$3056	0.17	W. D. Hatfield
Elyria, Ohio	25,000	460	155	1	850	750	$1200	3500	1.00	J. R. Collier
Urbana, Ill. (S. D.)	43,900	280	120	1	400	1200	75	1675	0.33	G. H. Radebaugh
Greensboro, N. C.	20,000	340	175	1	300	1200	60	1560[2]	0.28	G. W. Smedberg
Birmingham, Ala. (Shades Valley)	15,000	250	85	1	1500	350	275	2125	1.97	H. H. Hendon
Newark, N. Y.	8,000	250	30	1	450	275	57	783	0.15	T. J. Smith
Lima, Ohio	40,000	485	100	2	800	1000	200	2000	0.33	E. E. Smith
Aurora, Ill. (S. D.)	47,000	590	155	2	632	2550	1000	4182	0.62	W. A. Sperry
Hagerstown, Md.	28,000	290	84	1	872	740	60	1672	0.56	Fuller & McClintock
Durham, N. C. (No. S.)	30,000	800	75	1	1200	3500	700	5200[6]	1.00	W. M. Piatt
Akron, Ohio (No. 1)	200,000	675	145	2	5200	4000	1000	10200[3]	0.28	T. C. Schaetzle
Springfield, Ill. (S. D.)	70,000	550	190	2	3000	1600	400	5000	1.00	C. C. Larson
Schenectady, N. Y.	60,000	482	95	1	500	2500	500	3500[4]	2.00	Morris M. Cohn
Rockford, Ill. (S. D.)	75,000	760	175	2	1000	1300	250	2550	0.50	H. S. Merz
Peoria, Ill. (S. D.)	220,000	990	163	2	3230	2454	1300	7984[5]	0.45	L. S. Kraus
Baltimore, Md.	835,000	1200	115	6	4500	7000	300	11800	0.20	G. E. Finck
High Point, N. C.	37,000	960	190	2	2000	4000	1500	7500	0.90	E. H. Moss

[1] Includes chemicals.

[2] Used for both water and sewage works—Total pop. 58,000.

[3] Exclusive of land.

[4] Used as material testing laboratory by Department of Public Works, purchasing agent, etc.

[5] Equipped with bacteriological as well as chemical analyses and research.

[6] Includes recording gas calorimeter valued at $2000.

tion for a term of years, with a natural tendency to more equipment and chemicals than originally provided. For smaller plants with lesser requirements as to analytical work, the costs may be materially lower than indicated in Table I.

The author is inclined to regard $2,000 as a reasonable initial sum to allow for first class laboratory equipment, exclusive of building, suitable for the more useful sewage tests carried out by the "average" plant laboratory. The distribution would be, say $800 for furniture, tables and permanent fixtures, $1,000 for appartus, instruments, etc., and $200 for chemicals. A more pretentious schedule of analytical work might increase the allowance to $2,500. An expenditure in excess of $2,500 would imply considerable research or special work being intended. For such cases there is no definite upper limit.

The sums indicated above should not act as a deterrent to those operators of existing small plants who contemplate laboratory control. After all, the most essential information regarding sewage quality and plant efficiency is furnished by three determinations, to wit, (*a*) sludge solids, (*b*) suspended solids and (*c*) B.O.D. To obtain this information in quite satisfactory manner requires an outlay of not more than $300 to $500 if one is willing to forego store-bought benches and cabinets.

General Features of the Laboratory

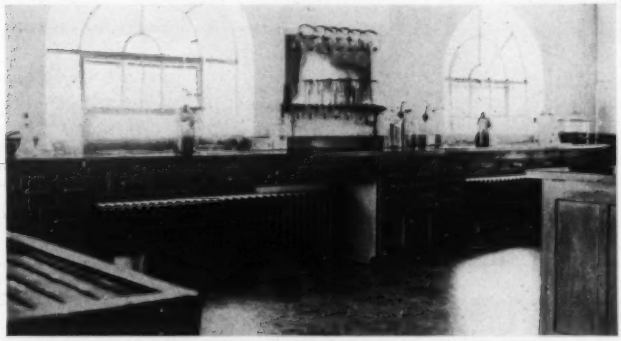
Location.—A laboratory to be most useful should be convenient to the plant it serves. If located in a room of the service building, as is customary, some study should be given to providing easy access to the plant itself on the assumption that the laboratory man may have to collect many samples and may be frequently called to various parts of the plant to observe operation. For the same reason, ground floor location for the laboratory is preferable to second floor. The northern exposure of the building is always to be preferred for laboratory location on account of the uniform light and absence of sunlight glare.

Floor Space.—From 250 to 300 sq. ft. is usually adequate where one worker is employed; where more than one worker uses the laboratory, more space is required, both to take care of the extra equipment and to avoid interference. The most convenient laboratory is one in which the fewest number of steps are required to carry out the operations. Large rooms are almost as undesirable as too cramped quarters. Aisle spaces between benches and apparatus should be not less than 4 feet, and preferably 5 feet.

Wall Space.—Wall space for placing cabinets, benches, hoods, incubators, etc., requires equal consideration with working floor space, and for that reason jogs or offsets of the interior walls may sometimes

work out better than a rectangular or square floor plan, because more wall space is made available.

Lighting.—Since laboratory operation calls for good vision, ample lighting is important. This means reasonable window space for daylight and ample artificial lighting for night work. In general, ceiling lights of the semi-direct type if properly placed, give ample lighting without throwing shadows across the benches so that bracket wall lights



F.G. 1.—Greater Peoria Sanitary District, Peoria, Illinois.

are unnecessary. Special lights over the balance and on the inside of the hood are desirable however.

Floors.—The author's preference is for rubber tile floor, although some object to rubber on the ground of being slippery if water is spilled. A good quality of linoleum is serviceable and easily kept clean. Hard wood floors are serviceable but less easy under foot than rubber tile, and not so easy to clean. Unyielding finishes like terrazo or ceramic tile are attractive in appearance but are quite tiresome to walk upon all day, also very destructive to glassware inadvertently dropped.

Wall Finish.—Probably the most practical wall finish is smooth plaster tinted lightly, or coated with white laboratory enamel if maximum lighting effect is desired. In some of the recent installations ceramic tile has been used effectively. Natural brick, either plain or pressed, has too much light absorption to be recommended. For the same reason any dark finish, either tile, paint or plaster, is to be avoided.

Benches and Tables.—The convenience of the laboratory is largely determined by the layout of permanent work benches, tables, hood, etc.

These should be installed with studied regard to the sequence of operation from the time samples arrive until they are disposed of. An attractive arrangement of tables, benches, cabinets, sinks, etc., without any consideration being given to the functions of these furnishings, may prove to be anything but convenient to the worker who uses them.

It would appear from Table I that about 100 square feet of work table and bench area is a fair allowance for one worker in a well equipped laboratory. Wall benches vary in width from 24 to 30 inches. The 30-inch width is preferable to allow for curbed window insets or cut-outs over the radiators. Center tables are customarily 4 feet in width

FIG. 2.—Akron Sewage Treatment Plant No. 1, Akron, Ohio.

to allow working space on both sides. Tables or benches used in a standing position should be 36 to 38 inches high above the floor, 37 inches being a fair compromise for most workers, while tables used in a seated position should be 30 to 32 inches high.

The choice between wall benches and center tables is usually governed by the floor space available. In general for a one man laboratory, rectangular in plan, having three or more outside windows, placing the main work bench against the outside wall and the hood, balance, etc., against the opposite interior wall provides the most convenient arrangement. For approximately square floor plans with limited outside window openings, center tables are best adapted, also where a full southern

exposure must be accepted. Center tables are useful when two or more workers use the laboratory space.

On the type of construction of the work benches and cabinets depends largely the cost of the laboratory furniture. It is usually cheaper in first cost to have the permanent furnishings installed by a local cabinet maker, but the furniture so made is seldom as satisfactory as articles furnished by a factory specializing in this type of construction. There in a wide selection of seasoned hardwood laboratory furniture on the market, either as complete fully equipped tables, or as standard cupboard and drawer sections that may be combined to suit the available building space. Work tops of bolted wooden strips stained with acid-proof aniline black, or tops of alberene stone slabs may be had, as preferred.

Preference is being shown in some new installations for table underbodies, cabinets, etc., of enameled steel fabrication with alberene stone tops. This type of furniture gives a pleasing appearance, is resistant to corrosion, while the drawers and cupboards work smoothly in damp as well as in dry weather.

Reagent Cabinet.—For storage of stock c.p. chemicals and standard solutions, a special cabinet is a practical necessity. A common mistake with chemical cabinets is to make them too deep so that that time is lost in searching for a desired reagent. A shelf width of not over 6 inches, preferably less, is desirable so that reagents may be arranged in a systematic order and in full view when the cupboard is open. Opaque doors instead of glass should be provided for the chemical cabinet as many chemicals and solutions deteriorate by continual exposure to light. The chemical cabinet should be provided with a good tumbler lock so that only the chemist has access, since there is a tendency towards unauthorized use of relatively expensive c.p. chemicals.

Fume Hood.—Although sewage laboratories are not notorious for chemical fumes, a good fume hood is a desirable item of the permanent furniture. Evaporation of moisture from sewage and sludge is usually in progress, and unless the vapors are vented from the room, they will produce odors and undesirable condensation on walls and windows, during winter months. Hoods of the open type with back baffle plate seem to be as serviceable as those of the sliding sash type and are cheaper and more accessible. Neither type of hood should be considered without a motor-driven exhaust fan to give positive ventilation. Where corrosive acid fumes are anticipated, a duriron or other acid-proof exhaust fan is to be recommended, with duriron or ceramic piping from the hood. Under ordinary conditions, however, less expensive fans with copper or monel piping give satisfactory service.

The fume hood should be of fireproof construction such as alberene

stone, transite or wire glass. A working top, 30 inches wide, of alberene stone, is quite satisfactory. The length will depend on local requirements and apparatus selected. A convenient combination under the hood is the steam bath, the drying oven and the muffle furnace. A Kjeldahl digestion battery, if provided, is best located in a separate compartment or a separate hood, although frequently mounted on an open table with separate exhaust manifold arrangement. There are available on the market complete hoods in various sizes and styles, with fan and fixtures to meet individual requirements.

Sinks and Plumbing.—For the laboratory used by a single worker, one centrally located sink will suffice as a minimum, but if more than one person uses the laboratory, there are sure to be traffic jams. Beside the main sink, it is desirable to provide a second sink devoted to washing sample bottles and glassware. The best sink is made of chemical stoneware with well rounded interior surfaces; it should be of a brownish tint which does not show stains readily. These sinks are more desirable than those made of cemented stone slabs as the stoneware sinks present no cracks or corners that are hard to keep clean. Sink drains inside the building should be of acid-proof material, such as duriron, stoneware or lead. In addition to the main working sink or sinks, it may be desirable to have one or more additional drain facilities such as 4 to

FIG. 3.—Aurora Sanitary District, Illinois.

6 inches diameter bowls of duriron, stoneware or lead set flush with the table top. The main sinks should be provided with both hot and cold water, supplied by a combination faucet with gooseneck extending high enough above the table to permit washing 1000 cc. cylinders.

Beside the sink faucets, cold water must be available at other points, as for instance, the water still, the steam bath, ammonia condensers, etc. It is better to consider these needs in specialized locations rather than to provide too many table fixtures which may never be used.

Balance Table.—It is customary to mount the analytical balance on a small table for use by the chemist in a sitting position. A common size top is about 20 inches in width and 4 feet in length, the extra length being desired for placing desiccators on either side of the balance. A simple alberene slab mounted on an open frame and leg support at a height of 30 to 32 inches is quite satisfactory. A drawer under the table may be provided to store analytical weights if desired, but should be shallow, else knee room may be cramped. There is seldom enough vibration to justify felt or cork pads on the table top.

Gas.—Gas, if available from either city mains or sludge digestion, should be piped to the principal work tables and the hood, with appropriate table fixture outlets, again with special reference to points where gas may be used.

Except for glass blowing, electric heat is suitable for practically all operations for which gas was formerly considered a necessity. Six laboratories in Table I use electricity for heating, nine use gas, seven use both, one uses bottled gas. Gas is not always available except as "bottled gas," hence the tendency to increased use of electrical appliances, for which suitable outlets should be provided on walls and tables. Current demands for some appliances are rather heavy, therefore it is necessary to provide independent circuits conforming to Underwriter requirements to such units as the electric water still, the steam bath, the muffle furnace, Kjeldahl digesters, and ammonia stills.

An air supply, under a few pounds pressure, is of little importance for routine work, although for special research work it is quite handy. Most activated sludge plants have an air supply readily available, otherwise if air is desired it must be supplied by a small motor-driven suction and blower unit.

The installation of vacuum piping connected to table cocks in various parts of the laboratory is hardly justified, since the use of vacuum is restricted almost entirely to Gooch filtrations which are carried out on one specialized section of the work table. Here a table or wall fixture is a desirable arrangement, comprising a vacuum manifold fitted with three-way cocks at proper height above the table to be connected to a battery of filter flasks. Vacuum may be supplied by an aspirator pump

or by a special motor-driven unit where many filter flasks are used simultaneously.

Special Shelving.—In addition to regular furniture such as tables and cabinets, it is the usual experience that some shelving to fit particular spaces and niches is required to keep the working tables clear of apparatus not in immediate use. The distilled water storage should be on a shelf about 30 inches above the main table and equipped with siphon and pinch cock for convenient use. This shelf should be located

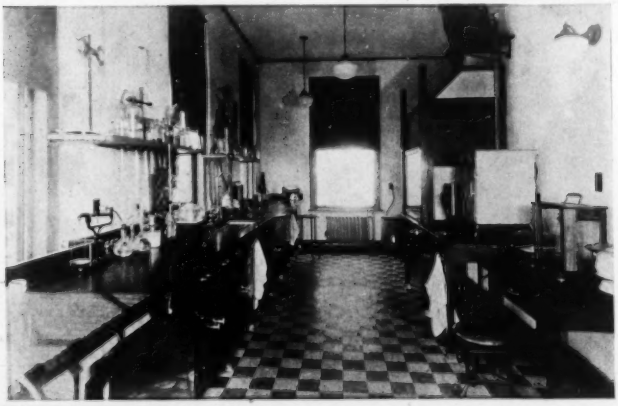


FIG. 4.—Hagerstown, Maryland.

above or adjacent to the main working sink. Diluting water for B.O.D. should be similarly supported. Various permanent supports and racks are required for special apparatus, the needs for which cannot be completely anticipated, and for which allowance should be made.

Stools.—Included in the laboratory furniture should be a couple of high stools for general use and one low stool dedicated to the balance table.

MOVABLE APPARATUS AND EQUIPMENT

No attempt will be made to discuss here the numerous items of apparatus and instruments, glassware, etc., that are more ore less common to all chemical laboratories, but brief mention will be made of a few important pieces of equipment.

Refrigerator for Samples.—Unless one is prepared to carry out a definite routine schedule for collecting composite samples at regular

FIG. 5.—Urbana and Champaign Sanitary District, Illinois.

intervals during the day, it is really futile to consider any extensive laboratory work aimed at evaluating sewage quality or plant efficiency. The sewage load is too variable throughout the day to allow a few random samples collected during the day shift to be a satisfactory measure of plant performance. Collection of composite samples means that these samples must be preserved from deterioration until subjected to laboratory analysis. Various preservatives may be applied to composite sample bottles where selected determinations are required, but if B.O.D. determinations are to be run, the only permissible preservative is low temperature refrigeration. For this reason an electric or gas refrigerator is a necessary adjunct to the laboratory although it may actually be located in some other part of the plant for the convenience of sample collectors.

B.O.D. Incubator.—The incubation of the B.O.D. tests has proved a serious stumbling block to the more general adoption of this valuable measuring stick. It is surprising that some important plants reporting B.O.D. results have no better incubation facilities than a selected corner of the basement where temperature variations may be several times the permitted tolerance. The B.O.D. determinations are too important to permit makeshift arrangements. There are on the market dependable 20° incubators, but they require an ice supply besides being unreasonably expensive. As a consequence the situation has been met in various

ingenious ways that point clearly to the need of a standardized piece of apparatus, offered at reasonable cost and ready for use in any location where electric current is available. The B.O.D. incubator arrangement used by representative laboratories responding to questionnaire are as follows:

Decatur, Ill.	200 lb. ice box, electrically heated, thermostat controlled. Capacity 42 bottles for B.O.D., 50 bottles for stability and 15 gal. carboy diluting water.
Elyria, Ohio	20° electric refrigerator-incubator combination, thermostat regulation.
Urbana-Champaign, Ill.	Ice box with thermo-regulators. Capacity 100 bottles.
Greensboro, N. C.	Electric water cooled with ice chamber. Capacity 35 bottles.
Birmingham, Ala.	Electric refrigerator with special control. Capacity 125 bottles.
Newark, N. Y.	Water tank 12 by 21 by 8 in. deep. Regulation by change of water.
Lima, Ohio	Water tank in electric refrigerator. Thermostatically controlled. Capacity 60 bottles.
Aurora, Ill.	Air incubator. Special type built in for 100 volt, 1 ampere. Two installed, each capacity 72 bottles on 3 shelves 12 by 16 inches.
Hagerstown, Md.	Insulated cabinet containing water tank, air stirred, and ice chamber. Regulating by needle valve control on water supply either direct, or through ice coils in summer.
Durham, N. C.	Electric refrigerator with 20° compartment built underneath, with thermostatic control and ice chamber.
Akron, Ohio	20° incubator (electrical heating and cooling unit), thermostatic temperature control, incubator equipped with 6 water tanks for submerging 8 oz. bottles during incubation. Capacity 250 bottles.
Springfield, Ill.	Electric incubator mounted inside an electric refrigerator. Capacity 60 bottles.
Schenectady, N. Y.	No description. Capacity, twenty 330 cc. bottles.
Rockford, Ill.	Electric refrigerator, 9 cu. ft. concerted into incubator using lights and thermostat to control temperature to 1° F. Capacity 35 bottles.
Peoria, Ill.	Electric refrigerator, 10 cu. ft. mercury thermometer regulator. Capacity 160 bottles in trays. Also three 2-gallon bottles.
Baltimore, Md.	4 by 5 ft. room, cork walls, mercoid control of ammonia and heaters. Capacity 800 bottles.
High Point, N. C.	Electric refrigerator mercury thermostat on inside heating element. Control 20° C. Capacity 100 bottles.

Conveniences and Time Savers.—The questionnaire showed some agreement on the following items as contributing most to convenience and time-saving:

1. Chainomatic balance for quick weighing.
2. Automatic burettes.
3. Gooch battery (vacuum manifold).
4. Stands and convenient diluting arrangements for B.O.D.

The author would be inclined to increase the above list to include

5. Centrifuge for rapid clarification of sewage and approximate determination of sludge solids.
6. Shallow nickel dishes for rapid evaporation of sludge samples.

Discussion

By Thomas M. Riddick

Instructor in Hydraulic and Sanitary Engineering, New York University

Mr. Donaldson has covered the subject so thoroughly and efficiently that anything I might say along this line would be superfluous. Mr. Donaldson courteously consented to my changing my topic from one of direct discussion of his paper to the inclusion of certain items of equipment not particularly mentioned by him, but described by me, in part, in the September, 1934, Journal of the American Water Works Association. (Pages 1238-1254.)

We have installed an efficient and economical 20° C. incubator in the laboratory of the Department of Hydraulic and Sanitary Engineering of New York University. No ice chamber is required and cooling is effected merely by the passage of water from city mains through a coiled copper tubing which surrounds the inside of the cabinet. Where the temperature of the city water rises above 20° C. in summer this line may be connected to a cooling element conveniently located in any standard refrigerator. A water seal is provided for the sample bottles and a short tube allows for the overflow of water from the tank upon the placement of samples. Water may be siphoned from the tanks for their removal when necessary. Of considerable facility is an inlet tap located above each tank. The capacity of the unit is 72 bottles and the cost considerably less than standard incubators now on the market.

Where chemical precipitation is used, and for the larger laboratories, a stirring apparatus may be found useful for determining the dosage of coagulant. The one we have installed was described in the J. A. W. W. A. article (q. v.). A Tyndall cone produced by the light placed under each beaker allows a ready determination of the minimum dosage required for a satisfactory floc.

In the interest of economy, or where no evaporation hoods have been initially installed in the laboratory, an inverted funnel may be suspended over the evaporating dishes on the steam bath, and a suction created by

an aspirator. Repeated analyses of fresh sewage sludge have demonstrated this to be a most effective device for the removal of odors.

A muffle furnace used in our laboratory has been calibrated so that a setting of 600° C. may be made for each ash determination thereby assuring uniformity of results.

Where several solids determinations are to be run at one time a dish and crucible tray will be found valuable in eliminating the possibility of confusing samples in the several weighings and desiccations required. It also provides a handy receptacle for the dishes upon their removal from the muffle furnace.

One of the most useful criteria for judging anaerobic sludge digestion is that of the composition of sludge gases. For this determination we have used a modified Orsat and a sampling devise which permits the collection of gas in an ordinary reagent bottle with easy transfer to the gas burette.

The presence of high grease content has a marked effect upon sludge digestion and is a factor of importance in the initial design of any plant. Ether-soluble greases and fats may be most accurately determined by the Sohxlet Apparatus. This accuracy, however, is generally unwarranted in sewage analyses. We have devised a simpler apparatus on which as many as six samples may be run at the same time. Ether is evaporated under suction created by an aspirator and no obnoxious fumes will be present in the laboratory.

Next in importance to the B.O.D. of the final sewage effluent is the dissolved oxygen content of the receiving stream. In making these determinations it is preferable to perform all analyses in the field rather than transporting samples to the laboratory, even after carrying them through to the release of iodine. We use a case for bottles, reagents and a burette, whereby the entire determination may be completed in the field.

In the general analyses of sewage Mr. Donaldson points out the great variation to be anticipated unless composite samples are collected throughout the entire 24-hour period of a day. We have found an equally great variation in fresh samples, due to the non-homogeneous nature of both sewage and sludge. Many discrepancies due to non-homogeneous samples can be eliminated by the simple expedient of passing the sewage through an ordinary meat grinder or fine screen, thereby ensuring a more equal distribution of all organic particles.

TREATMENT OF SEWAGE AT OKLAHOMA CITY WITH IRON, CHLORINE AND LIME *

By L. H. Scott

Chemist, Sewage Treatment Works, Oklahoma City

The Northside Sewage Plant of Oklahoma City treats all the sewage from the northwest section of the city, which amounts to some 4.5 million gallons per day. The sewage is entirely domestic and contains an average of 200 p.p.m. of suspended solids. The plant is some eight miles from the city and with the small amount of grade to the sewers and with the high summer temperature the sewage reaches the plant in a septic condition and high in hydrogen sulphide. To make the problem more acute, the effluent discharges into a small stream that is entirely dry during the summer months, and flows through a territory that is fast becoming, at least in part, a district of the better country homes and clubs.

The original plant was put into operation in July of 1927 and consisted of two Imhoff tanks designed to handle two million gallons per day (Fig. 1). This plant was not completed to the extent of any sec-

Fig. 1.—Northside Sewage Treatment Plant, Oklahoma City.

ondary treatment other than a contact tank for the application of final chlorine. The total cost of the installation was $86,748.

The plant gave fair satisfaction but was soon greatly overloaded due to the boom period of Oklahoma City during the development of the oil field, and the fact that the section of the city connected to the Northside Plant soon became the largest residential section of the city. The

* Presented at the Joint Meeting of the Arkansas-Oklahoma Water Works and Sewage Works Conference, Fayetteville, Ark., April 16, 1935.

Imhoff tanks were not able to handle the additional load and were continually foaming with the result that the creek during the summer months received such a load from the plant that numerous lawsuits were filed and won against the city. This condition existed for several years, continually growing worse until it became necessary for Oklahoma City to build a plant that would be able to correct this condition.

From observation of the condition of the creek it was felt that the plant would have to remove the settleable suspended solids completely as the accumulation of these solids along the creek banks had in the past been most noticeable and objectionable. With this in mind it was natural to turn to chemical precipitation. The progress of the Dearborn plant was closely watched and during the time it was being constructed, experiments were being made to determine if this process would be suitable for use in Oklahoma City.

As a result of these experiments a decision was made to go ahead with a similar process, but at that time ferric chloride, the main chemical used in the process, and the one shown to be by far the best for the purpose, could not be obtained at a price that would permit us to use it in Oklahoma City.

At the same time chlorine was being purchased in Oklahoma City in ton containers at a price of $2.80 per hundred pounds and this naturally led to the manufacture of ferric chloride by the use of chlorine and iron in an endeavor to get this chemical at such a price that it could be used in the treatment of the sewage.

Considerable time was spent in the development of equipment for the manufacture of the ferric chloride, as we then desired a piece of equipment that would not only make the chemical from chlorine and iron turnings, but would also regulate the dosage. This eliminated the use of the batch method and the use of storage tanks and regulators for the feeding of the chemical. After some time a piece of equipment was designed to accomplish what we wanted, and then we found out how little we really did know about the simple chemical ferric chloride, for we found it possible to make solutions that would have different characteristics, and of still more importance, different coagulating qualities. It was finally determined that the difference was caused by the amounts of iron taken into the solution by the action of the chlorine and that, with certain rates of flow through the reaction tank and with certain strengths of chlorine solution, it was possible to make a final ferric chloride solution containing a great quantity of colloidal iron. This we found to be the best coagulant, and then it was necessary to redesign the equipment so that this solution could be made at all times and fed to the sewage in the proper proportions.

At this time I want to thank the Oklahoma A. & M. College for the work done and the help they gave us in finally running down and finding out the exact composition of the material we were making. Drs. Smith and Trimble, of the Chemistry Department, spent considerable time and devised some very interesting equipment for the demonstration of the various reactions brought about by the action of chlorine solution on iron.

The plant as it stands now completed is, I might say, a good example of progressive development. We started out to do one thing but in trying to accomplish that one thing so many other reactions developed that before the problem was cleared up we had just about completed a course in chemistry, with all the necessary apparatus for the demonstration of the reactions on a plant scale.

Our first attempt at chemical precipitation, after the equipment for the manufacture of the ferric chloride had been installed, was at the then existing Northside Plant. Immediately we found that, due to the high concentration of the hydrogen sulphide, the ferric chloride was broken down to ferrous sulphide. This eliminated the odor of the hydrogen sulphide gas about the plant, but as ferrous sulphide was a very poor coagulant, it did not accomplish what we set out to do, namely, eliminate the solids in the effluent of the plant. Several methods for overcoming this were tried and finally the plant took on the form shown on the flow sheet (Fig. 2).

The sewage enters through a Lakeside Comminutor, which not only acts as a screen, but also cuts up all the screenings so that they will pass through a three-eighths inch mesh. This in reality does away with the screenings and the necessity of handling them, as they are then considered as settleable solids and handled in the plant in the same manner.

Just after being screened and comminuted, the sewage is treated with lime to raise the pH to 8.5. This is done for the dual purpose of precipitating the ferrous sulphide formed and also for conditioning the sludge prior to filtration. Immediately following the application of the lime, ferric chloride is added, the treated sewage is mixed for a period of twenty minutes in the flocculator, and then allowed to settle in the two primary clarifiers. These are equipped with drag flights and the settled sludge is continuously removed and pumped to the vacuum filter for dewatering. The effluent from the primary clarifiers is then chlorinated and allowed to settle in the two secondary clarifiers, also equipped with similar drags to remove the settled solids.

The sludge from both the primary and secondary clarifiers is pumped continuously to the vacuum filter and there dewatered (Fig. 3). The sludge cake is either given to the farmers for fertilizer or hauled and dumped into a nearby ravine.

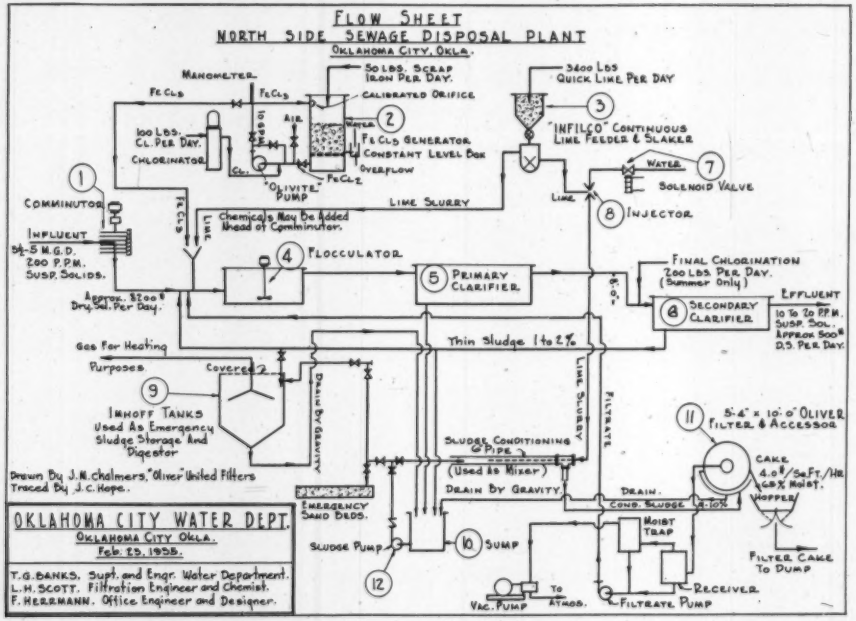

FLOW SHEET
NORTH SIDE SEWAGE DISPOSAL PLANT
OKLAHOMA CITY, OKLA.
MANOMETER
50 LBS. SCRAP IRON PER DAY.
CALIBRATED ORIFICE
3600 LBS QUICK LIME PER DAY
"INFILCO" CONTINUOUS LIME FEEDER & SLAKER
FeCl3
100 LBS. CL. PER DAY.
CHLORINATOR
AIR
WATER
FeCl3 GENERATOR
CONSTANT LEVEL BOX
OVERFLOW
"OLIVITE" PUMP
WATER
SOLENOID VALVE
COMMINUTOR
LIME SLURRY
LIME
INJECTOR
Chemicals May Be Added Ahead of Comminutor.
INFLUENT
5 M.G.D.
200 P.P.M.
SUSP. SOLIDS.
FLOCCULATOR
PRIMARY CLARIFIER
FINAL CHLORINATION
200 LBS. PER DAY.
(Summer Only)
EFFLUENT
SECONDARY CLARIFIER
10 TO 20 P.P.M. SUSP. SOL.
Approx. 500# D.S. PER DAY.
Approx. 8200# Dry Sol. Per Day.
Thin Sludge 1 to 2 %
Gas For Heating Purposes
Covered
IMHOFF TANKS
Used As Emergency Sludge Storage And Digestor
DRAIN BY GRAVITY
LIME SLURRY
FILTRATE
5'-6" x 10'-0" OLIVER FILTER & ACCESSOR
SLUDGE CONDITIONING PIPE
(Used As Mixer)
CAKE
4.0#/SqFt./HR.
65% Moist.
HOPPER
DRAIN.
DRAIN BY GRAVITY
COND. SLUDGE
4-10%
EMERGENCY SAND BEDS.
MOIST TRAP
SLUDGE PUMP
SUMP
FILTER CAKE TO DUMP
RECEIVER
VAC. PUMP
TO ATMOS.
FILTRATE PUMP
Drawn By J.N. Chalmers, "Oliver" United Filters
Traced By J.C. Hope.
OKLAHOMA CITY WATER DEPT.
OKLAHOMA CITY OKLA.
Feb. 23, 1935.
T.G. BANKS. Supt. and Engr. Water Department.
L.H. SCOTT. Filtration Engineer and Chemist.
F. HERRMANN. Office Engineer and Designer.

FIG. 2.

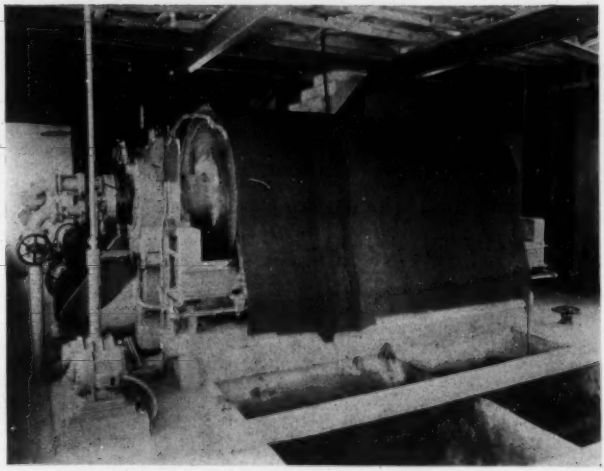


FIG. 3.—Oliver Vacuum Filter, Oklahoma City.

The Imhoff tanks were covered and converted into digestors and enough sludge is pumped into them to furnish gas for the heating of the building during the winter months. This digested sludge is then dried on the drying beds of the original plant, as this sludge would require further conditioning with chemicals before it could be filtered on the vacuum filter.

The plant was designed for a dry-weather flow of 5.0 million gallons per day, and is now completed with the exception of the installation of an additional vacuum filter. This will permit the filtering of all of the sludge during the day shift and reduce somewhat the operating expenses of the plant.

While the completed plant has been in operation only three months, the work has been going on for the past three years, so some idea of the results obtained can be had over the entire period while the operating cost will have to be taken only over the few months that the plant has been operating as a complete unit.

For the past two years the suspended solids in the raw sewage have averaged 200 p.p.m. and these have been reduced to an average of 10 or less in the effluent. The B.O.D. in the raw sewage averages 185 p.p.m. and in the effluent 19 p.p.m. (Table I). The cost of the treatment for

TABLE I.—*Treatment of Sewage at the North Side Sewage Plant, Oklahoma City, Okla.*

	Per Day	Per Month
Average Flow	5.5 M.G.	171.6 M.G.

Treatment	Grains/Gal.	Per M.G.	Per Day
Lime	5.27	753 lbs.	4145 lbs.
$FeCl_3$	.35	51 "	280 "

	Solids	
Character of Sewage and Effluent	Raw	Effluent
Total Solids	897 p.p.m.	712 p.p.m. ($CaCO_3$).
Volatile	317 p.p.m.	155 p.p.m.
Suspended	200 p.p.m.	10 p.p.m.
Settleable	8.3 c.c./liter	0.0 c.c./liter
	B.O.D., 5-day	
	185 p.p.m.	19 p.p.m.

Operation of Vacuum Filter	
Sludge per M.G.	
Wet, Cu. Ft.	397
Dry, Lb.	2500
Filter Rate, 5.5 Lb. per Sq. Ft. per Hr.	

the last three months, including wages, salary, power, maintenance, chemicals, and overhead, has been $8.04 per million gallons of sewage treated. The cost of the chemicals used during this period averaged $4.08 per million gallons (Table II).

The vacuum filter was manufactured by the Oliver United Filter Company and has an area of 165 square feet. The sludge is pumped from the clarifiers with an average of 10.0 per cent solids and has been dewatered at a rate of 5.5 pounds of dry solids per square foot per hour. The average moisture of the filter cake is 62 per cent. An average of 2500 lb. of dried solids have been filtered per million gallons of sewage treated for the last two months.

In summing up the operation and treatment, I would say that the cost is reasonable. The effluent is clear and the condition of the creek is perfect, with no unsightly solids and no odors. The sludge from the filter is stable, and even though piled in the ravine where it has been rained on, has never given any trouble from odors. The plant is under cover, is fly-proof and easily operated and controlled. Most of the equipment is automatic and all under the supervision of one operator.

Some of the problems encountered in trying to get the process developed turned out to be helpful if they were put to work in the right manner and if applied at the right points. The fact that the hydrogen

TABLE II.—*Cost of Treatment at the North Side Disposal Plant,* Oklahoma City, Okla.*

	Per Month	Per Year
Chemicals		
Lime @ $10.90 per ton	$ 542.27	$ 6507.24
Chlorine @ 2.80 per 100 lbs.	130.20	1562.40
Total—Chemicals	$ 672.47	$ 8069.64
Supervision and Labor		
Supervision	$ 41.00	$ 492.00
Labor		
1 @ $125.00	125.00	1500.00
1 @ 100.00	100.00	1200.00
1 @ 90.00	90.00	1080.00
2 @ 3.20 per day	192.00	2304.00
Total—Labor and Supervision	$ 548.00	$ 6576.00
Transportation		
1 Truck—Full Time	$ 38.66	$ 463.92
Supplies		
Filter Clothes	$ 20.73	$ 248.76
Miscellaneous Supplies	6.00	72.00
Total—Supplies	$ 26.73	$ 320.76
Utilities		
Power	$ 44.15	$ 529.70
Water	8.75	105.00
Total	$ 52.90	$ 634.70
Total Operating Cost	$1379.76	$16,557.02

* These figures are based on the three months of operation of the North Side Plant. The average flow per month handled has been 171.6 M.G.

sulphide immediately reacted with the ferric chloride to form ferrous sulphide led to the use of this chemical for the treatment of sewage for the elimination of the hydrogen sulphide, which had been giving trouble in the sewers from the packinghouses in Oklahoma City. Hydrogen sulphide had attacked the concrete until several of the larger sewers had fallen in, necessitating expensive repairs. If this condition had been allowed to continue, it would have meant the replacing of the sewers throughout the city. Chlorine had been tried as a remedy, and while it is true theoretically that the amounts of chlorine figured either way, that is as chlorine or as ferrous chloride, should neutralize the same amounts of hydrogen sulphide, we found that as the sewage became stronger in organic matter it would take more and more of the chlorine for this purpose, the chlorine being absorbed by he organic matter. In the packinghouse wastes so much chlorine was required to do the work that the costs were prohibitive.

By adding ferrous chloride, the condition was reversed, as none of this chemical is absorbed by the organic matter and all is used for the purpose of converting the hydrogen sulphide to ferrous sulphide. This is a stable product as long as the sewage does not become acid. It quite successfully eliminated the problem we had facing us in protecting the sewer line. One other advantage of this form of treatment is that it is possible to carry a residual of ferrous chloride in the sewage, so that if the sewage should develop hydrogen sulphide or if some sewage high in hydrogen sulphide should enter into the line below the application of the ferrous chloride, there will be sufficient of the chemical as a residual to counteract and eliminate the hydrogen sulphide. This property permits applications of ferrous chloride at any point along the line on the main interceptor or the sewers that require protection, without the additional equipment necessary with chlorine

At the present time there are some twelve plants installed using this process and several more are let and are ready for the start of construction.

EXPERIMENTS ON SETTLING AND FILTERING ACTIVATED SLUDGE AERATED LIQUORS

NORTH SIDE TREATMENT WORKS, SANITARY DISTRICT OF CHICAGO *

By S. I. Zack

Sanitary Engineer, Filtration Equipment Corp., 10 E. 40th St., New York, N. Y.

General.—The Filtration Equipment Corporation by agreement with the Sanitary District of Chicago installed a Laughlin Tank consisting of a 3 ft. wide Magnetite Filter on a 20 ft. diameter steel tank, for the purpose of testing the applicability and efficiency of a Magnetite Filter operating on activated sludge aeration tank liquors (Fig. 1). This tank was operated and laboratory analyses made by the Sanitary District. The Filtration Equipment Corporation cooperated in the supervision of these tests. The tests covered a period from about July 1, 1934 to February 9, 1935 and up to 1.1 m.g.d. of aeration tank liquor was handled.

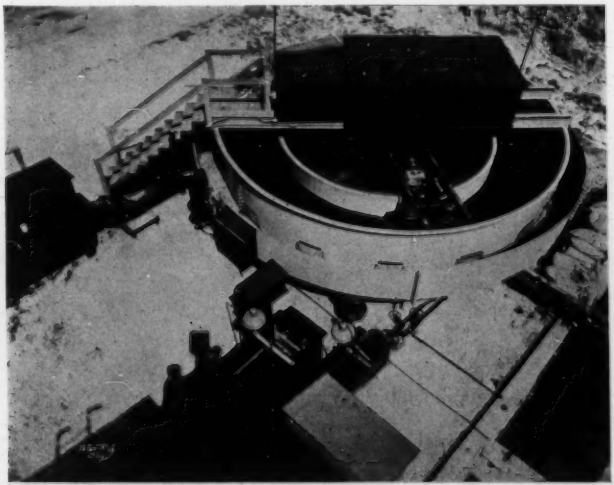


Fig. 1.—Laughlin Tank at North Side Plant, Chicago. Showing 20-foot tank, 3-foot magnetite filter and washer and measuring boxes.

* Presented before the Spring Meeting of the New York Sewage Works Association, Poughkeepsie, May 24, 1935.

Location of Tests.—The Laughlin Test Plant was located at the northwest end of Battery A, Aeration Tanks, North Side Sewage Treatment Works of the Sanitary District of Chicago. Activated sludge mixed liquor was taken from Tank A-12 in Battery A (Fig. 2).

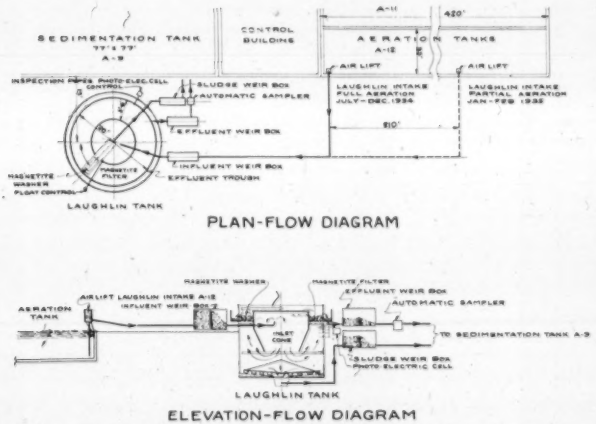


FIG. 2.—Plan and elevation showing general layout of Laughlin Test Plant, North Side, Chicago.

Purpose of Test.—The purpose of this test was to determine the following principal data and information pertaining to the magnetite filter for application to final activated sludge settling tanks:

1. Whether activated sludge had properties detrimental to the mechanical operation of the magnetite filter and washer.
2. The quality of effluent obtainable, operating difficulties and control necessary with the magnetite filter installed on final tanks operating on fully and partially aerated activated sludge liquors:
   (*a*) At settling rates equal to mechanically cleaned settling tanks without filters.
   (*b*) At settling rates higher than mechanically cleaned settling tanks without filters.
   (*c*) At various rates of filtering in combination with low and high settling rates.

   (The Sanitary District design rates of 1200 gal. per sq. ft. per 24 hr. at average and 1950 gal. per sq. ft. per 24 hr. at storm

flow conditions for aeration tank liquors discharged to final settling tanks are the rates considered equal to mechanically cleaned settling tanks throughout this paper.)

3. Design features for most efficient operation and results, such as:
   (*a*) Proportions of settling areas, sludge concentration volume and magnetite filter areas required.
   (*b*) Loss of head through filter bed at various rates of flow and conditions of solids beneath the filter.
   (*c*) Control and operation of filter bed washer.
   (*d*) Control of sludge blanket.

The Sanitary District was principally interested in the magnetite filter as applied to tanks at the higher settling and filtering rates to produce an effluent equal in quality to that produced by mechanically-cleaned settling tanks without filters. Therefore, most of the work was concentrated on this basis. Those tests of specific immediate interest to the District were designated as witness tests, although the same routine and procedure in the field and laboratory were also carried out for the early tests designated as preliminary and later, on the partial aeration tests.

Such improvements that could be made simply by changes in the tank itself or its operation were made from time to time. Other improvements recognized as necessary were limited by the particular design of the fixed features of this tank and could not be made. Under certain conditions the results obtained were undoubtedly handicapped by the inability to make these changes, but in such cases wherever possible, data was obtained to indicate how correctives could be incorporated in a new design.

Description of Test Plant.—The Laughlin test plant included the following tanks, equipment, apparatus and auxiliaries:

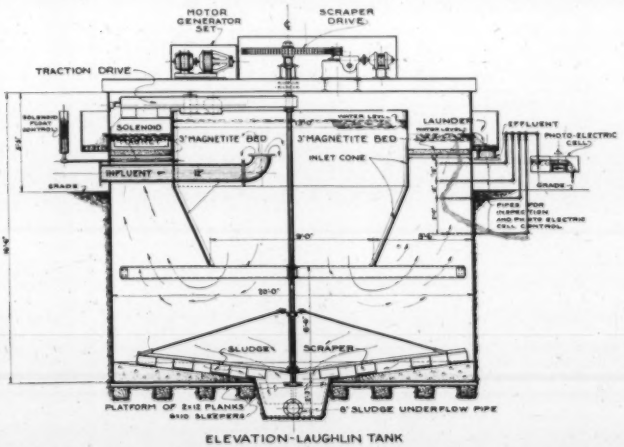
A steel settling tank 20 ft.–0 in. in diam. 14 ft. water depth at the side with the bottom sloping to a central sludge collecting sump (Fig. 3). This tank was fitted with sludge scraping mechanism driven through a central shaft and a 3 ft. wide magnetite filter around the periphery. The sludge scraping mechanism was operated continuously at a speed of one revolution in 20 minutes.

The magnetite filter (Fig. 4) consisted of a bed of magnetite, from 10 to 20 mesh, 2½ to 3 in. deep, retained on an Everdur phosphor bronze screen supported on steel bars 2½ in. apart. Mechanism for washing the filter consisted of a Laughlin Magnetite Filter washer composed of a traction driven travelling caisson carrying a solenoid in one compartment and a wash-water return pump in the other.

The washing mechanism and traction drive started automatically by a float switch when the loss of head through the bed increased due to accumulation of solids in the voids of the magnetite filter.

Operation of the sludge scrapers and washing mechanism were separately controlled. The caisson containing the solenoid and wash-water pump was operated by direct current supplied from a 10 H.P. motor-generator set and travelled at a speed of 3 ft. per minute. The sludge mechanism was operated by alternating current.

Washing operation was effected by the solenoid energized and de-energized through limit and contactor switches making and breaking the current to the solenoid. When the current went into the solenoid, it picked up a section of sand about 4 in. wide on a radius across the bed.



ELEVATION-LAUGHLIN TANK

FIG. 3.—Elevation showing details of Laughlin Tank.

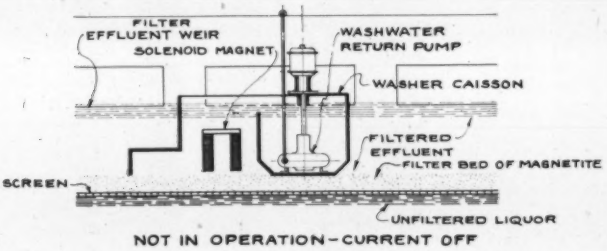
This released the pressure through the bed at that section, causing liquors in the top portion of the settling tank just below the magnetite filter to rush up into the solenoid caisson. Then, when the current went off the solenoid, the dirty magnetite dropped through this water. The wash water flowed over a baffle into the pump suction chamber, also a part of this caisson. The wash water return pump kept the elevation of water in the pump suction chamber below the level of the water in the solenoid portion, thereby causing the dirty wash water to continually flow in the direction of the pump chamber. This difference in head prevented the wash water from getting back into the clear effluent through which the entire caisson travelled.

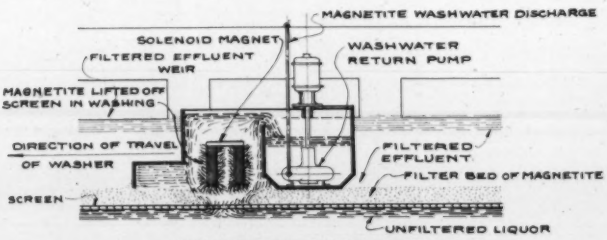
An air-lift pump was used for supplying activated sludge mixed liquor to the settling tank equipped with a magnetite filter. Separate

weir boxes, equipped with flow gauges and Bristol recorders, were used for measuring flows of mixed liquor, effluent and return sludge.

An automatic sampler was used on the effluent line for sampling.

A photo-electric cell and auxiliaries were used for controlling the sludge elevation and operating the 8 in. motor-driven sludge return valve. The photo-electric cell was operated by flowing liquor from the tank below the filter through a 1½ in. pipe at one of four set elevations





MAGNETITE WASHER

FIG. 4.—Details of Magnetite Washer.

over an electric eye. At intervals of 7½ minutes, if the liquor was dirty, the return sludge valve would open, and when the liquor flowing past the electric eye was clear, the valve would close.

A recording meter was used for measuring electrical consumption.

A U-tube manometer, with the high pressure side connected to the inlet cone of the tank and the low pressure side connected to the liquor above the filter, was used for measuring the loss of head through the filter.

Operating Control.—The test plant was operated continuously 24 hours per day by the shift force of the Sanitary District along with the regular operation of the North Side Plant. General supervision was given cooperatively by representatives of the Sanitary District and Filtration Equipment Corporation.

Aerated activated sludge mixed liquor was pumped through a weir box into the central portion or inlet cone of the Laughlin tank. The sludge settled at the bottom and was discharged by gravity through a weir box. The effluent was filtered upward through the magnetite filter and discharged through a weir box. Wash water from the filter was discharged to the inlet of the Laughlin tank.

A continuous record of flow was obtained on the mixed liquor influent, final effluent and sludge return. Independent inspections and readings were also made manually, and recorded by the plant operators, of the head over the mixed liquor, effluent and sludge return weirs, loss of head over filter, turbidity from appearance of liquid coming out of the inspection pipes at various depths below the filter, air used and flow in aeration tank A-12, from which the mixed liquor was drawn.

Power consumption was recorded by a meter on a continuous chart, and also from integrator readings on each shift.

Recording flow charts were changed at midnight and run from midnight to midnight.

Sampling.—Samples of effluent for suspended solids determination were taken automatically with a mechanical sampler, every four minutes, sufficient to fill a 2-gal. bottle in 24 hours. The sludge return and wash water were sampled by hand hourly and composited for 24 hours. In addition to the mechanical sampling, the effluent was sampled by hand and composited, for each 8-hr. shift, in gallon bottles which were combined into a single 24-hr. sample for analysis in the laboratory. Hand samples were also taken of the liquor 6 in. below the filter composited for each 8-hr. period in gallon bottles, but due to press of laboratory routine, these samples were analysed only occasionally. At various times, as a check, hourly samples were also taken of mixed liquor.

Samples were composited for the 24 hours from 8 A.M. to 8 A.M. This was done to correspond with the plant routine.

For a number of days samples of sludge were also taken at various depths in the clarifier every two hours during the day shift and composited.

Twice each shift, sludge settling tests were made in 1000 cc. graduates, recording the volume occupied by the settled sludge at the end of one hour. Once a day, a settling test was made to determine the speed of settling in 1000 c.c. graduates.

Laboratory Analyses.—The 24-hr. composites were taken to the North Side laboratory for analyses by the Sanitary District chemists at the beginning of the morning shift. Suspended matter determinations were made on the mixed liquor, wash water, effluent and occasionally of the liquor taken below the filter. Total solids were determined in the sludge underflow. At first B.O.D. analyses were made of a sample taken over 24 hours by the mechanical sampler. It was found that seeding and nitrification occurred in the sampling lines and sampler, causing an increase in the B.O.D. This method was discontinued and analyses were later made on samples taken by hand, put on ice at the end of each shift and composited into one 24-hr. sample in the laboratory at the end of three shifts for B.O.D. analysis.

Operating Features of North Side Plant as Affecting Aerated Liquor Handled by Laughlin Tank.—The average sewage flow to the North Side Plant was about 200 m.g.d. Approximately 0.4 cu. ft. of air per gal. of sewage was applied and the aeration period was 5 hours. Pumps and air blowers were usually reduced during the night with consequent increase after midnight and during the day until the interceptor was pumped down to normal.

As near as could be controlled, the air and detention time in Tank A–12 was maintained the same as in the rest of the battery. The aerated liquor from A–12 to the Laughlin tank, however, was taken at a uniform rate. A–12 also received its proportion of return sludge from the return sludge channel in Battery A.

Operating and Analytical Data.—Data are reported according to the various test periods. The aerated activated sludge liquor handled in the Laughlin tank varied not only in its solids content but also relative to its settling characteristics, which affected the rates of operation and sludge underflow differently under dry weather and storm flow conditions. By the division of tests into periods, it was attempted to study the effects and limitations of as many variables as could be practically determined.

The work and results are divided into Preliminary Periods "A," "B" and "C" from July 3 to Aug. 9, 1934. Witness test periods 2, 3, 4 and 5 from August 10 to Nov. 16, and a test on partial aeration from Jan. 3 to Feb. 10, 1935.

It is hardly possible in the scope of this paper to report all of the data obtained, therefore, a brief outline of pertinent information and tabulations of principal daily operating and analytical data and as summarized by periods are included in the following:

Settling rates are calculated on the basis of 314 sq. ft., which is the entire surface area of the tank, including the magnetite filter, and reported as gallons of aeration tank liquor influent to Laughlin tank per

square foot of tank surface per 24 hours. Filtering rates of effluent in gallons per sq. ft. per min. are based on the net area of 160 sq. ft., which is obtained by deducting from the gross filter area, the filter area lost by angle supports and grout seal of the screen on the outside and inside rims.

Preliminary Periods "A," "B" and "C."—July 3 to Aug. 9, 1934. Sampling and records were started July 3, 1934. Satisfactory results were obtained at settling rates up to 2280 gallons per sq. ft. per 24 hours. An attempt was made to start the first witness test July 15th at higher rates. Results obtained were unsatisfactory for rates above 2400. Witness test No. 1 was discontinued. After a suitable stilling well was installed to quiet the disturbance in the inlet cone, and an inspection pipe was placed 30 in. below the filter to observe the sludge blanket, it was possible to get satisfactory results at rates up to about 3000. The results averaged into three periods according to rates of approximately 1400, 2100 and 2800 gal. per sq. ft. per 24 hrs. are shown in Table I. Rates of 2400 were attempted prior to proper stilling arrangements of the influent, but the results were unsatisfactory.

Both the B.O.D. and suspended solids averaged 5 p.p.m. or less for rates up to 2100 and 7 p.p.m. at rates up to 2800.

Witness Test No. 2.—Aug. 10 to Sept. 12, 1934. Witness Test No. 2 was set up to carry through the program of testing, started in the first test, discontinued on July 15 to make adjustments in the inlet of the Laughlin tank. The results are averaged for the entire period, as well as for the period from August 10 to September 2, and September 3 to September 12 separately. This divides the operation into so-called "intermediate rates" which might occur in an activated sludge plant for a month at a time and into "storm rate." The Laughlin Tank was operated at an average rate of 2760 from August 10 to September 2 and at an average of 3320 between Sept. 3 and 12.

It was difficult by manual operation to adjust the sludge underflow to meet the variable characteristics of the influent at high rates to prevent a rapidly rising sludge blanket elevation. If the underflow valve was throttled or clogged sludge accumulated in the Laughlin tank to a point above the filter causing sludge to go through it and affecting the quality of the effluent. A photo-electric cell furnished by the Sanitary District was therefore installed Aug. 17 to control automatically the opening and closing of the sludge underflow valve according to a selected elevation of the sludge blanket. The cell first operated on a 15-minute cycle and later on a 7½-minute cycle. Until the photo-electric cell was properly adjusted, sludge occasionally went through the filter. Such days were reported, but averages with and without such days are shown.

TABLE I.—*Daily Results of Operating and Analytical Data Settling-Filtering Activated Sludge Fully Aerated Liquor in Laughlin Tank*

1934 Date	Influent gal. per sq. ft. per 24 hrs.	Pct. Under-Flow	Filter Rate gal. sq. ft./ min.	Filter Eff.	
				Susp. Solids	B.O.D.
Preliminary Period "A" Rates of Flow—1300 to 1800 gal. per sq. ft./24 hrs.					
July					
3	1605	28.4	1.70	3	2.7
4	1465	31.5	1.53	3	3.5
5	1465	37.4	1.45	5	3.4
8	1800	22.4	2.00	7	4.3
9	1502	23.1	1.66	—	4.8
10	1335	27.0	1.43	5	3.8
11	1448	29.7	1.53	5	2.7
12	1431	28.2	1.54	4	2.1
13	1780	18.3	2.05	4	4.9
Av.	1534	27.4	1.65	4	3.6
Preliminary Period "B" Rates of Flow—2000 to 2200 gal. per sq. ft./24 hrs.					
6	2020	31.0	2.10	6	4.5
22	2050	45.0	1.94	11	4.3
23	2050	23.8	2.26	6	2.9
24	2080	21.3	2.33	5	3.9
25	2145	16.4	2.51	5	4.0
26	2200	32.7	2.26	5	4.3
28	2150	37.7	2.13	6	3.5
29	2100	50.0	1.91	6	2.3
30	2050	32.7	2.11	4	13.5
31	2115	19.2	2.41	1	4.8
8/1	2130	16.1	2.50	4	6.2
Av.	2100	29.6	2.23	5	4.9
Preliminary Period "C" Rates of Flow—2300 to 3100 gal. per sq. ft./24 hrs.					
8/2	2575	29.6	2.70	4	3.7
3	3020	67.0	2.47	—	—
4	2320	20.6	2.63	13	9.6
5	2390	19.7	2.73	7	5.0
6	2625	23.7	2.89	7	7.5
7	2900	21.5	3.25	4	4.8
8	3100	26.9	3.34	4	8.3
9	2930	25.4	3.18	5	7.5
Av.	2740	23.9	2.90	6	6.6

1934 Date	Influent gal. per sq. ft. per 24 hrs.	Pct. Under-Flow	Filter Rate gal. sq. ft./ min.	Filter Eff.	
				Susp. Solids	B.O.D.
Witness Test No. 2					
Aug.					
10	2815	24.7	3.07	6	12.2*
11	2830	23.2	3.12	6	3.3
12	2755	21.7	3.10	16	17.0*
13	2815	21.1	3.18	8	16.7*
14	2815	19.5	3.20	6	4.3
15	2770	19.0	3.16	7	3.8
16	2815	15.0	3.31	8	4.5
17	2870	18.3	3.30	5	6.2
18	2870	11.0	3.56	10	7.0
19	2845	16.4	3.32	7	4.7
20	2830	11.1	3.47	7	5.1
21	2755	12.9	3.09	5	6.0
22	2870	18.5	3.30	7	5.9
23	2990	26.3	3.22	13	7.8
24	2870	34.2	2.92	33*	15.4*
25	2640	27.5	2.81	13	12.7
26	2405	24.0	2.64	22*	14.6*
27	2625	22.4	2.92	24*	17.2*
28	2815	25.2	3.05	14	11.5
29	2640	35.1	2.66	12	20.4*
30	2545	38.2	2.51	10	7.9
31	2260	27.0	2.43	12	10.4
Sept.					
1	2670	40.0	2.61	8	14.8
2	2930	39.1	2.86	9	10.7
3	3200	30.6	3.34	10	12.8
5	3500	37.2	3.46	6	5.9
6	3400	34.0	3.46	8	3.5
7	3200	34.2	3.25	5	6.3
8	3370	45.8	3.15	12	13.3
9	3400	45.0	3.19	9	7.8
10	3180	28.6	3.38	7	8.8
11	3265	31.0	3.40	8	8.2
12	3310	36.2	3.33	7	6.4
Av. 8/10–9/2	2760	28.8	3.04	11	10.0
Av. 9/3–9/12	3320	26.0	3.33	8	8.1
Av. 8/10–9/12	2910	27.1	3.12	10	9.5

Another inspection pipe 46 in. below the filter was installed to aid control of the sludge blanket during this period.

An average effluent containing less than 10 p.p.m. B.O.D. and suspended solids was obtained during this period. At the high rates in the Laughlin tank a rate of sludge underflow higher than for ordinary settling tanks at average rates was required when sludge had poor settling characteristics.

The daily operating and analytical results are shown in Table I.

Witness Test No. 3.—Sept. 13 to Oct. 2, 1934. This period was divided into two parts during which rates of flow averaged 2180 from Sept. 13 to 21 and 3030 from Sept. 22 to Oct. 2. The concentration of influent liquor increased to 3000 p.p.m. and averaged 2700 p.p.m. An effluent averaging 10 p.p.m. or less suspended solids and B.O.D. was obtained. During the first part of the period (3A), sludge underflow averaged 19.6 per cent whereas in the latter part (3B), the underflow was excessive. Period 3B represented a combination of conditions caused by high rates, more concentrated influent liquors and poorer settling sludge than previously. Sludge samples taken at various depths in the Laughlin tank indicated considerable stirring up, unsatisfactory concentration and stratification of solids. These data are reported as a separate study in a portion of this report to follow. Table II shows the results for Period 3.

Witness Test No. 4.—Oct. 2 to 16, 1934. After several days of period 4A the sludge mechanism was found to be down, and the traction wheel on the washer caisson had to be replaced. During 4B the sludge blanket was allowed to hit the filter frequently by operating the photo-electric cell controlling the sludge underflow from an inspection pipe 6 in. below the filter. This was to approximate conditions that might result from a bulking sludge. The average effluent during this period was not so good as previously. Although the head loss through the filter increased somewhat, it did not upset the self-cleaning features of the washer mechanism. Table II shows the results for this period.

Witness Test No. 5.—Oct. 19 to Nov. 16, 1934. During this period 50 per cent of the filter area was blanked off with wooden plates. The

Notes: July 16 to 21 incl.—Started Witness Test No. 1 but discontinued to adapt stilling well and baffling for high rates.

* If these days are omitted averages and maximums shown in Table "A" are obtained. Reasons for omission: Aug. 10 and 13 on account of sampling. Other days not considered representative of close sludge blanket control.

Table "A"

	Susp. Sol.	B.O.D.
Av. Aug. 10 to Sept. 2	9	7.5
Av. Aug. 10 to Sept. 12	9	7.7
Max.	14	14.8

TABLE II.—*Daily Results of Operating and Analytical Data Settling-Filtering Activated Sludge Aerated Liquor in Laughlin Tank*

1934 Date	Influent gal. per sq. ft. per 24 hrs.	Pct. Under-Flow	Filter Rate gal. sq. ft./ min.	Filter Eff. Susp. Solids	Filter Eff. B.O.D.
		Witness Test No. 3A			
Sept.					
13	2220	24.8	2.43	5	3.8
14	2595	31.5	2.69	6	10.5
15	2160	16.9	2.52	7	10.7
16	2220	18.9	2.55	5	8.2
17	2200	19.8	2.51	6	9.5
18	2145	13.8	2.58	6	7.7
19	2080	15.6	2.46	7	9.3
20	2100	13.3	2.52	10	9.6
21	1940	21.4	2.18	7	10.5
Av.	2180	19.6	2.50	7	8.9
		Witness Test No. 3B			
22	2725	56.0	2.39	8	11.9
23	3230	130.0	1.92	7	7.6
24	3230	86.0	2.37	7	9.0
25	2450	79.0	1.87	11	7.8
26	3065	59.0	2.64	9	13.1
27	3570	103.0	2.40	8	5.6
28	3500	101.0	2.36	23	12.6
29	3165	79.0	2.42	8	20.0
30	3050	87.3	2.22	19	20.7
Oct.					
1	2770	54.2	2.48	6	8.5
2	2563	54.0	2.26	8	7.7
Av.	3030	81.0	2.30	10	11.3
		Witness Test No. 4A			
4	3165	130.0	2.25	14	8.5
5	3056	70.0	2.43	5	9.6
6	2870	47.0	2.66	7	10.3
7	2520	55.0	2.22	17	19.1
Av.	2903	76.0	2.39	11	11.9

Date	Influent gal. per sq. ft. per 24 hrs.	Pct. Under-Flow	Filter Rate gal. sq. ft./ min.	Filter Eff. Susp. Solids	Filter Eff. B.O.D.
		Witness Test No. 4B			
Oct.					
10	2275	63.3	1.90	9	11.3
11	2305	36.5	2.30	16	16.7
12	2145	31.9	2.22	20	14.8
13	2180	37.3	2.17	8	12.3
14	2210	44.2	2.10	5	11.0
15	2100	29.5	2.21	7	5.6
16	2115	25.3	2.36	7	8.6
Av.	2190	34.1	2.18	10	11.5
		Witness Test No. 5A			
19	2115	22.8	4.1	10	14.8
20	2220	31.9	4.2	24	22.5
25	1940	31.7	3.5	13	12.5
26	2260	38.3	3.9	13	12.6
27	2360	34.5	4.2	15	14.7
28	2340	34.9	4.2	14	12.6
30	2180	35.9	3.9	11	11.0
31	1955	29.3	3.6	12	10.3
Nov.					
1	2690	42.5	5.1	12	9.2
2	2130	42.3	4.1	11	11.9
3	2305	37.8	4.6	10	9.1
Av.	2254	34.1	4.2	13	12.4
		Witness Test No. 5B			
4	1575	24.4	3.4	9	10.3
5	1400	15.3	3.3	5	7.3
6	1400	21.3	3.1	6	14.8
7	1400	22.7	3.1	15	7.6
8	1352	21.6	3.0	9	6.6
12	1400	37.0*	2.8	9	5.3
13	1400	18.4	3.2	9	8.6
14	1400	16.7	3.3	7	7.5
15	1368	19.5	3.1	8	5.2
16	1715	18.5	4.0	12	13.0
Av.	1441	21.5	3.2	9	8.6

Notes: Test No. 3B—Excl. Sept. 28 and 30—Average B.O.D. 10.2, Susp. Solids 8 P.P.M.
4A—Sludge scraping mechanism not in operation.
4B—Until Oct. 15 operated with sludge blanket close to filter to determine effect of allowing sludge to get into filter frequently.
5—Fifty per cent of filter area blanked off with wooden plates.
* Too high minimum setting on photo-electric cell.
More sludge than required was drawn.

filter rate averaged 4.2 gal. per sq. ft. per min. for period 5A, and 3.2 for 5B. An average of 5.2 gal. per sq. ft. per min. for an entire day, with maximums of approximately 6 gal. per sq. ft. per min. for short periods, was satisfactorily reached. Head loss through the filter at these rates was not excessive. Average effluents containing less than 15 p.p.m. B.O.D. and suspended solids were obtained. The average rates of flow were 2254 and 1441 gal. per sq. ft. per 24 hrs. for periods A and B respectively. Results for this period are shown in Table II.

Partial Aeration Tests.—Jan. 3 to 20, 1935 and Feb. 6 to Feb. 10, 1935. The Laughlin tank was operated on partially aerated activated sludge liquors taken from a point half way down Aeration Tank A–12. Since fully aerated sludge was returned to Tank A–12 the results of this test cannot be considered conclusive as to the specific reduction of aeration time possible when followed by settling-filtering instead of settling alone. The tests were run primarily to obtain information with the facilities available and are indicative that further work in this direction is worthwhile. A logical application would be the installation of a magnetite filter on an overloaded plant in lieu of extension.

Satisfactory performance at an average rate of 1410 and a maximum of 2160 gal. per sq. ft. per 24 hrs. was obtained. The corresponding filter rates were—average 1.40 and maximum 2.30 gal. per sq. ft. per min. The effluent contained 11.0 p.p.m. B.O.D. and 16 p.p.m. suspended solids. Perhaps more significant than the actual effluent obtained were the analyses of samples taken 6 inches below (before filtering) and above (after filtering), showing removals by the filter, shown in another part of this paper (Table IX), and the action of the filter and washer mechanically on what may be termed an underaerated activated sludge liquor.

A summary of the results obtained on the partially aerated liquor is shown in Table III.

Summary of Operating and Analytical Data.—The principal operating and analytical data are summarized by periods in Table III.

Averages are adjusted omitting the several days not considered representative of suitable sludge blanket control. Sludge blanket control at the high rates of operation was not satisfactory until the photo-electric cell was installed and it was learned how to adjust its performance. Variable factors in the photo-electric cell itself requiring adjustment were light intensity, increment of underflow sludge valve opening and time interval of cell function, which had to be adjusted to such variables as flow, sludge characteristics and sludge elevation in the tank.

The filter was operated at average rates of 1.4 to 4.2 gal. per sq. ft. per min. and average settling rates of 1410 to 3320 gal. per sq. ft. per

TABLE III.—*Summary of Operating and Analytical Data*
Settling-Filtering Activated Sludge Mixed Liquor in Laughlin Tank

Test	A-12 Aeration Tank Data			Laughlin Tank Influent	Sludge Underflow		Filter Rate	Susp. Solids			5-Day B.O.D.		
	Cu. Ft. Air/Gal.	Aer. Per. Hrs.	Pct. Sol. M.L.	Gal. Sq. Ft. per 24 hrs.	Pct. of Eff.	Pct. Sol.	Gal. Sq. Ft. per Min.	Raw Sewage	Eff.	Pct. Red.	Raw Sewage	Eff.	Pct. Red.
						Full Aeration							
A	.35	5.7	.21	1534	27.4	1.20	1.65	136	4	97	110	3.6	97.0
B	.40	5.1	.24	2100	29.6	1.23	2.23	141	5	97	111	4.9	95.8
C	.42	5.0	.26	2740	23.9	1.42	2.90	141	6	96	108	6.6	94.0
No. 2*A*	.41	6.0	.21	2760	23.8	1.34	3.04	131	9*	93	104	7.5*	92.6
2*B*	.46	5.5	.21	3320	36.0	0.84	3.33	154	8	95	119	8.1	93.0
3*A*	.40	5.6	.27	2180	19.6	1.72	2.50	173	7	96	113	8.9	92.4
3*B*	.38	5.2	.27	3030	81.0	0.63	2.30	142	8*	94	113	10.2*	91.0
4*A*	.36	5.3	.25	2903	75.5	0.71	2.39	104	11	90	106	11.9	88.8
4*B*	.38	5.4	.21	2190	34.1	0.85	2.18	142	10	93	138	11.5	91.8
5*A*	.35	5.5	.19	2254	34.1	0.72	4.20	164	13	92	155	12.4	92.0
5*B*	.44	6.5	.20	1441	20.0*	1.43	3.20	146	9	94	142	8.6	93.8
						Partial Aeration							
	.15	2.7	.22	1410	27.7	1.61	1.40	140	16	89	99	11.0	89.0

* Adjusted averages omitting days not considered representative of close sludge blanket control (see daily results).

General Notes

Tests *A*, *B* and *C* cover period from July 3 to August 9.
Test 3*B*—Test at high rates and high suspended solids in aerated liquor in influent to Laughlin Tank.
Test 4*A*—Sludge mechanism not in operation.
Test 5—50 per cent of Magnetite Filter blanked off.
Partial Aeration—Influent to Laughlin tank taken half way down *A*-12, but *A*-12 received fully aerated return sludge from entire Battery A.
Air and detention time is for portion of aeration tank up to Laughlin intake.

24 hrs. Activated sludge liquor aerated from 5 to 6 hours with 0.35 to 0.44 cu. ft. of air per gallon was handled in the Laughlin tank. During periods 4A and 4B at rates of about 3,000 gal. per sq. ft. per 24 hrs. the percentage underflow was excessive.

Average effluents containing from 4 to 13 p.p.m. suspended solids and 3.6 to 12.4 p.p.m. B.O.D. were obtained, depending on the rate of settling and filtering.

Satisfactory performance of the filter was obtained with under-aerated liquor to which was returned fully aerated sludge.

Solids Handled and Sludge Concentration in Laughlin Tank.—It is apparent that in the combined settling-filtering in a Laughlin tank of liquor having the characteristics of activated sludge the problem resolves itself into (1) the suitable concentration of the solids which can be withdrawn in a reasonable quantity of underflow and (2) the filtering of the supernatant liquor which would normally be discharged over a weir in an ordinary final settling tank. Undoubtedly, there is a most efficient point where the sludge settling and concentration should end and the filtering begin, which could be provided in a design by a proper proportioning of tank area and volume and magnetite filter area.

The effects of rates of influent and sludge concentration were studied from (1) calculations of the pounds and concentration of solids handled in the Laughlin Clarifier at various rates of flow and (2) from analyses of sludge samples taken at various depths in the tank.

Table IV shows the flows of influent and sludge correlated with the amounts and concentration of solids as averaged for the periods.

TABLE IV.—*Average Solids Handled in Laughlin Tank*

Test No.	Aerated Liquor Influent		Underflow		Pounds per Day Dry Solids Handled	Sludge Index *	Pct. Volatile Susp. Solids	
	Rate	Pct. Solids	Pct. of Effluent	Pct. Solids			Raw Sewage	Aerated Liquor
A	1534	0.21	27.4	1.20	8300	67	69	70
B	2100	0.24	29.6	1.23	13700	73	73	71
C	2740	0.26	23.9	1.42	18100	66	59	71
2A	2760	0.21	23.8	1.34	16000	54	76	68
2B	3320	0.21	36.0	0.84	18500	66	66	69
3B	2180	0.27	19.6	1.72	15400	54	65	64
3C	3030	0.27	81.0	0.63	21500	66	69	67
4A	2903	0.25	75.5	0.71	16100	67	79	69
4B	2190	0.21	34.1	0.85	11700	82	73	73
5A	2250	0.19	34.1	0.72	10200	81	74	74
5B	1441	0.20	20.0	1.43	9500	72	80	75

* Based on 1 hr. settling in 1,000 c.c. graduates.

Table V shows the results of special analyses of samples at various depths from the bottom at the rates of influent for those particular days.

TABLE V.—*Analysis of Depth Samples in Laughlin Tank*

Date	Influent		Pct. Solids At Depths from Bottom in Ft.				Weighted Average Pct. Solids in Tank
	Rate	Pct. Solids	.67	3.0	6.0	9.0	
Sept.							
19	2080	0.25	2.36	1.10	0.75	0.16	0.97
20	2100	0.26	1.80	0.90	0.63	0.16	0.79
21	1940	0.23	1.60	0.79	0.32	0.11	0.62
22	2725	0.28	0.77	0.37	0.19	0.20	0.35
23	3230	0.29	0.44	0.25	0.25	0.21	0.27
26	3065	0.25	0.33	0.15	0.22	0.15	0.20
27	3570	0.27	0.29	0.20	0.21	0.19	0.21
28	3500	0.26	0.45	0.20	0.20	0.19	0.24
Oct.							
1	2770	0.31	0.47	0.25	0.22	0.23	0.29
2	2563	0.28	0.62	0.80	0.27	0.23	0.47
6	2870	0.24	2.42	0.40	0.16	0.16	0.34
11	2305	0.22	0.82	0.20	0.24	0.13	0.30

Table VI shows the detention time of the sludge in the Laughlin tank for the various rates of flow calculated from the solids influent and analyses of depth samples.

TABLE VI.—*Detention Time of Sludge in Tank*

Date	Influent Rate	Pounds Dry Solids Added per Hour	Pounds Solids in Tank	Detention of Sludge in Hours	Underflow	
					Pct. of Effluent	Pct. Solids
Sept.						
19	2080	625	2085	3.34	15.6	2.22
20	2100	585	1689	2.88	13.0	2.07
21	1940	542	1330	2.46	21.4	1.58
22	2725	750	740	1.00	58.0	0.64
23	3230	940	584	0.62	130.0	0.41
26	3065	790	435	0.55	58.1	0.63
27	3570	1050	455	0.43	102.0	0.55
28	3500	940	512	0.55	108.4	0.44
Oct.						
1	2770	953	629	0.66	52.1	0.86
2	2563	790	1002	1.27	54.0	0.85
6	2870	780	717	0.92	47.2	0.77
11	2305	500	646	1.29	36.5	0.76

The depth samples indicate that at high rate of flow, especially when large amounts of solids were going into the Laughlin tank, considerable disturbance and stirring prevented good stratification of solids and consequently affected the concentration. The degree of disturbance and effect on concentration of course varied, and was dependent on the settling characteristics of the sludge.

It was believed that the disturbances were to a certain extent fixed by the hydraulic elements, and that better sludge concentration would be obtained in a tank having a greater ratio of surface to depth than was possible in the 20 ft. diameter by 14 ft. deep tank being tested. Therefore, three tests were run in an attempt to determine, at high rates of influent flow, a comparison of sludge settling characteristics between the 20 ft. Laughlin tank and the 77 ft. square large North Side settling tanks. The results shown in Tables VII and VIII might be interpreted to indicate better concentration in large tanks, even though conclusions

TABLE VII.—*Tests at High Rates Comparing Large 77 Ft. Tanks with 20 Ft. Laughlin Tank*

	Influent		Underflow		Filter Rate	Susp. Solids	
	Rate	Pct. Sol.	Pct. of Effluent	Pct. Solids		6″ Below Filter	Final Effluent
1st Test—3 hours duration							
77 ft.	3800	.19	27.5	0.71	—	—	100
20 "	3165	.19	45.6	0.46	5.2	151	15
2d Test—23½ hours duration							
77 ft.	4000	.18	*20.5+	0.66	—	—	38
20 "	3020	.18	49.2	0.64	3.3	55	18
3d Test—39 hours duration							
77 ft.	3080	.19	*30.5+	0.79	—	—	37
20 "	2990	.19	44.7	0.78	3.3	45	8

* Does not include sludge passed to main drain.

TABLE VIII.—*Comparison of Depth Samples—Per Cent Solids*

Ft. from Bottom	1st Test		2d Test		3d Test, 2d Day	
	77 ft.	20 ft.	77 ft.	20 ft.	77 ft.	20 ft.
9	.27	.15	.14	.13	.13	.16
6	.21	.20	.16	.15	.28	.17
3	.34	.21	.20	.10	.35	.22
0.67	.78	.15	.53	.37	.67	.70

from a quantitative standpoint cannot be specifically drawn from them on account of difficulty in controlling the operation of the large tanks at twice their rated capacity.

Since the sludge concentration seems to be better in large tanks and the filter took out an amount of solids nearly equivalent to that which went over the large tank weirs, it is to be expected that better results could be obtained with a filter on a larger tank than was obtained with the 20 ft. tank.

Removals by Magnetite Filter.—During part of the tests, samples were drawn through a 1½ inch pipe 6 in. below the filter extending 12 in. inside the tank. Analyses were made for comparison with the filtered effluent to determine the removals by the filter.

Table IX shows suspended solids for 22 days during the period on fully aerated liquor between Sept. 13 and Dec. 29, 1934, and the partially aerated liquor for 21 days between Jan. 3 and Feb. 10, 1935.

TABLE IX.—*Removals by Magnetite Filter*

Date	Settling Rate Gal./sq. ft. per 24 hr.	Filter Rate Gal./sq. ft. per min.	Suspended Solids	
			6 in. Below Filter	Filtered Effluent
		Fully Aerated Liquor		
Sept. 13	2220	2.4	13	5
14	2595	2.7	16	6
16	2220	2.6	25	5
18	2145	2.6	28	6
19	2080	2.5	128	7
20	2100	2.5	24	10
25	2450	1.9	27	11
27	3570	2.4	38	8
Oct. 4	3165	2.3	26	14
5	3056	2.4	23	5
6	2870	2.7	52	7
7	2520	2.2	60	17
Nov. 15	1368	3.1	16	8
16	1715	4.0	14	12
17	2450	5.4	18	11
18	1975	4.0	14	15
23	1210	3.0	49	10
24	1335	2.1	22	12
25	1225	2.3	18	12
Dec. 15	3020	3.7	55	18
28	2900	3.3	51	7
29	3028	4.2	23	11
Average	2328	3.0	34	10

TABLE IX.—*Continued*

Date	Settling Rate Gal./sq. ft. per 24 hr.	Filter Rate Gal./sq. ft. per min.	Suspended Solids	
			6 in. Below Filter	Filtered Effluent
Partially Aerated Liquor				
Jan. 3	1210	.9	17	9
4	1210	1.2	22	12
5	1210	1.3	25	18
6	1240	1.0	47	14
7	1225	.9	29	16
8	1431	1.5	21	18
9	1431	1.5	26	18
10	1275	1.4	22	16
11	1240	1.0	32	11
14	1210	1.4	17	10
15	1210	2.0	27	10
16	1560	2.0	34	18
17	1975	2.2	47	18
18	2160	2.5	36	15
19	2050	2.6	51	14
20	1880	2.3	106	19
Feb. 6	1225	1.3	36	28
7	1115	1.5	31	24
8	1230	1.5	23	14
9	1305	1.5	35	14
10	1225	1.6	51	16
Average	1410	1.6	35	16

The suspended solids 6 in. below the filter in the series on fully aerated liquor ranged from 13 to 128 p.p.m. averaging 34 p.p.m. and the filtered effluent ranged from 5 to 18 p.p.m. averaging 10 p.p.m. In the series on partially aerated liquor the range was from 17 to 106 p.p.m. 6 in. below the filter, averaging 35 p.p.m.; and the effluent between 9 and 28 p.p.m. averaging 16 p.p.m. suspended solids.

Head Loss Through Filter.—Measurements to determine the head loss through the magnetite filter bed were made at intervals. This was done by means of a manometer, one leg connected to the inlet well of the tank representing the pressure below the filter, and the other leg to the clear liquor compartment over the magnetite.

The loss of head obtained at these intervals are plotted against rate of flow through the filter in Fig. 5.

The first measurement was made July 13 and repeated July 16 after backwashing with a one-inch hose. The magnet, found to be too high to pick up all the magnetite for washing, was lowered 1½ in. July 17.

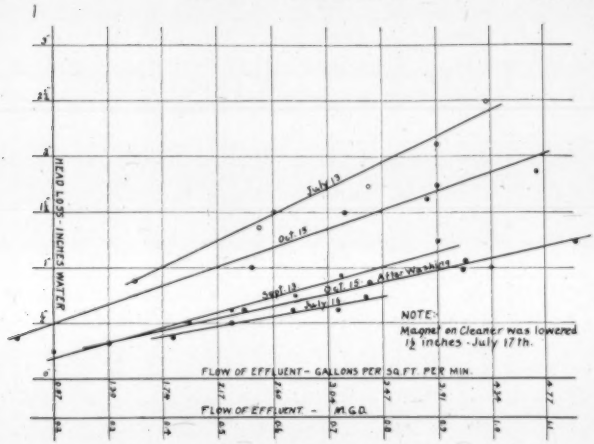

HEAD LOSS - INCHES WATER
July 13
Oct. 15
Sept. 13
Oct. 15 - After Washing
July 13
NOTE:
Magnet on Cleaner was lowered
1½ inches - July 17th.
FLOW OF EFFLUENT - GALLONS PER SQ. FT. PER MIN.
FLOW OF EFFLUENT - M.G.D.

FIG. 5.—Curves showing loss of head in inches of water for various flows and filter rates.

The Sept. 15th measurement was made after the filter has been operating intermittently at a rate of 3.0 gal. per sq. ft. per minute. On October 15th the measurement was made after operating the sludge blanket for a week either in the filter or just below. Backwashing with a 1½ in. hose cleared the filter to its original rating. The losses of head as shown by these tests ranged between ¼ and 2½ inches for filter rates between 1 and 5 gal. per sq. ft. per minute.

Filter Wash Water.—The test plant was designed for either discharging filter wash water to the sludge underflow or returning it back to the inlet cone of the Laughlin tank. Except for several runs of short duration showing no improvement in performance, normal operation consisted of returning the wash water back to the influent of the tank.

The wash water pump had a rated capacity of 50 gal. per min. but was usually operated with the discharge throttled. No actual measurements were made but the filter wash water pumped back was estimated at 3.0 to 10 per cent of the total daily influent flow to the tank. The rate depended on the filter rate, solids filtered, head loss and time of operation of the washer, which ranged from 12½ to 100 per cent of the time.

These percentages of wash water would not apply directly for tanks of different ratios of tank diameter and filter width.

Suspended solids in daily wash water samples varied from 33 to 1100 p.p.m., normally averaging from 200 to 300 p.p.m.

Conclusions

The following conclusions may be drawn from a comprehensive study of the Laughlin magnetite filter and tank tests on settling-filtering activated sludge liquors and pertain to conditions similar to those tested:

1. The magnetite filter and washer is mechanically adaptable to filtering partially and fully aerated activated sludge liquors.
2. Effluents attainable on the North Side sewage were:
   a. 4 to 5 p.p.m. suspended solids and B.O.D. for ordinary standard settling and filter rates of 2 gal. per sq. ft. per min.
   b. 8 to 10 p.p.m. suspended solids and B.O.D. for settling rates up to 3500 gal. per sq. ft. per 24 hrs. combined with filter rates of about 3.0 gal. per sq. ft. per min. Operation at higher than standard rates was limited by the sludge concentration capacities required which depend on settling and concentration characteristics and amount of the solids in the influent liquors, in any case requiring close control of the sludge blanket. Large tanks may afford more suitable hydraulic conditions than small ones.
3. Filter rates of 6, for short periods, 5 for a period of a day, and averaging 4 gal. per sq. ft. per min. are practical for magnetite filters on activated sludge liquors but effluents under 15 p.p.m. B.O.D. or suspended solids cannot be expected under these conditions.
4. Positive removals by the filter of suspended solids up to about 100 p.p.m. on individual days and averaging 20 to 25 p.p.m. over a period are possible.
5. Loss of head through the filter bed is not excessive, normally reaching a maximum equivalent to 3 inches of water.
6. The filter can be designed to provide protective features acting as a safety valve to counteract underactivation, poor settling conditions or bulking sludge.

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Stream Pollution

MEASURES OF NATURAL OXIDATION IN POLLUTED STREAMS. II. THE REAERATION FACTOR AND OXYGEN BALANCE.

By H. W. Streeter

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In the first part of this paper,* the various influences have been described which modify progressive changes in the oxygen demand of sewage-polluted streams and methods have been indicated by which their effects may be evaluated from the results of laboratory tests for biochemical oxygen demand (B.O.D.) on the stream water. The second part which follows will deal with the role of atmospheric reaeration in maintaining a reserve supply of dissolved oxygen in streams to support natural oxidation, and with the effects of combined deoxygenation and reaeration on the trend of progressive changes in their dissolved oxygen content.

In addition to atmospheric reaeration, or absorption of oxygen directly from the atmosphere, another important source of oxygen in natural bodies of water exists in the photosynthetic action of certain oxygen-producing plants, which, under favorable conditions, may exert a tremendous influence on the rate of oxygen recovery by such waters. As this phase of the subject will be discussed in a forthcoming paper by Special Expert W. C. Purdy of the Public Health Service, it will not be considered here, except to note that atmospheric reaeration probably is a more widespread and consistent source of oxygen in natural bodies of water than its biological prototype; hence, must be regarded as the more important of the two, from a practical viewpoint.

Nature and Laws of Atmospheric Reaeration

Whenever an open body of water is undersaturated with dissolved oxygen, it will absorb oxygen from the overlying atmosphere at a rate dependent on (1) the partial gas pressure of oxygen in the atmosphere; (2) the degree of oxygen unsaturation or saturation deficiency existing in the water at its surface which may vary to some extent with the water temperature; (3) the area of water surface exposed, and (4) the rapidity with which fresh particles of undersaturated water are brought to the surface and in contact with the atmosphere. It is probable that hydro-

* This Journal, **7**, 251 (1935).

diffusion and what Adeney [1] has termed "streaming" exert a powerful influence on the distribution of oxygen to the lower water strata, though in flowing streams physical agitation, or turbulence, is also a very important factor.

In Table I are given the oxygen saturation values, or maximum amounts of oxygen, in parts per million, dissolved by water under a

TABLE I.—*Saturation Values of Dissolved Oxygen, in Parts per Million, for Temperatures in ° C. Ranging from 0° C. to 30° C.*

(Under Normal Atmosphere at 760 mm. Pressure)

Temp. (° C.)	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	14.62	14.58	14.54	14.50	14.46	14.42	14.39	14.35	14.31	14.27
1	14.23	14.19	14.15	14.11	14.07	14.03	14.00	13.96	13.92	13.88
2	13.84	13.80	13.77	13.73	13.70	13.66	13.62	13.59	13.55	13.52
3	13.48	13.44	13.41	13.38	13.34	13.30	13.27	13.24	13.20	13.16
4	13.13	13.10	13.06	13.03	13.00	12.97	12.93	12.90	12.87	12.83
5	12.80	12.77	12.74	12.70	12.67	12.64	12.61	12.58	12.54	12.51
6	12.48	12.45	12.42	12.39	12.36	12.32	12.29	12.26	12.23	12.20
7	12.17	12.14	12.11	12.08	12.05	12.02	11.99	11.96	11.93	11.90
8	11.87	11.84	11.81	11.79	11.76	11.73	11.70	11.67	11.65	11.62
9	11.59	11.56	11.54	11.51	11.49	11.46	11.43	11.41	11.38	11.36
10	11.33	11.31	11.28	11.25	11.23	11.21	11.18	11.15	11.13	11.11
11	11.08	11.06	11.03	11.00	10.98	10.96	10.93	10.90	10.88	10.86
12	10.83	10.81	10.78	10.76	10.74	10.71	10.69	10.67	10.65	10.62
13	10.60	10.58	10.55	10.53	10.51	10.48	10.46	10.44	10.42	10.39
14	10.37	10.35	10.33	10.30	10.28	10.26	10.24	10.22	10.19	10.17
15	10.15	10.13	10.11	10.09	10.07	10.05	10.03	10.01	9.99	9.97
16	9.95	9.93	9.91	9.89	9.87	9.85	9.82	9.80	9.78	9.76
17	9.74	9.72	9.70	9.68	9.66	9.64	9.62	9.60	9.58	9.56
18	9.54	9.52	9.50	9.48	9.46	9.44	9.43	9.41	9.39	9.37
19	9.35	9.33	9.31	9.30	9.28	9.26	9.24	9.22	9.21	9.19
20	9.17	9.15	9.13	9.12	9.10	9.08	9.06	9.04	9.03	9.01
21	8.99	8.98	8.96	8.94	8.93	8.91	8.89	8.88	8.86	8.85
22	8.83	8.81	8.80	8.78	8.77	8.75	8.74	8.72	8.71	8.69
23	8.68	8.66	8.65	8.63	8.62	8.60	8.59	8.57	8.56	8.54
24	8.53	8.51	8.50	8.48	8.47	8.45	8.44	8.42	8.41	8.39
25	8.38	8.36	8.35	8.33	8.32	8.30	8.28	8.27	8.25	8.24
26	8.22	8.20	8.19	8.17	8.16	8.14	8.13	8.11	8.10	8.08
27	8.07	8.05	8.04	8.02	8.01	7.99	7.98	7.96	7.95	7.93
28	7.92	7.90	7.89	7.87	7.86	7.84	7.83	7.81	7.80	7.78
29	7.77	7.75	7.74	7.73	7.71	7.70	7.69	7.67	7.66	7.64
30	7.63									

normal atmosphere at 760 mm. pressure and at temperature intervals of 0.1 degree centigrade ranging from 0° C. to 30° C. This table has been inserted merely for purposes of general reference.

From a practical standpoint, the most important factor influencing the rate of reaeration is the oxygen saturation deficiency in the underlying body of water, which very largely determines this rate under a given set of physical conditions. The results of Adeney's studies [1] have led him to express the law of reaeration in the form of the following mathematical equation:

$$w = w_1 - (w_1 - w_0)e^{-f(a/v)t}$$

or

$$w_1 - w = (w_1 - w_0)e^{-f(a/v)t} \qquad (1)$$

in which

w = concentration of dissolved oxygen at the end of time (t),
w_c = concentration at the start, when $t = 0$,
w_1 = saturation concentration at the given temperature and pressure,
f = a coefficient of escape of the gas from the water per unit of area,
v = volume of water,
a = surface area exposed,
e = the Naperian logarithmic base (= 2.7183 approximately).

In Adeney's equation, it will be noted that $(w_1 - w_0)$ is the initial degree of unsaturation, or saturation deficiency, of oxygen in the water and $(w_1 - w)$ the corresponding deficiency after time (t), both being expressed in terms of percentage of saturation or any other unit of concentration.

In connection with a study of oxidation and reaeration in the Ohio River, Streeter and Phelps [2] derived a similar expression for the rate of atmospheric reaeration from the simple differential equation:

$$-\frac{dD_t}{dt} = K_2 D_t \qquad (2)$$

which states that the rate of decrease $(-(dD_t/dt))$ in oxygen saturation deficiency is directly proportional to that deficiency (D_t) at any time (t). The integrated form of this equation gives the expression:

$$D_t = D_a \cdot e^{-K_2 t} \qquad (3a)$$

or

$$D_t = D_a \cdot 10^{-k_2 t} \qquad (3b)$$

in which $K_2 = 2.3k_2$ and (D_a) is the initial oxygen saturation deficiency.

The close analogy between equation (3a) and Adeney's formula (1) may be readily shown by noting that (D_t) is equivalent to $(w_1 - w)$, (D_a) to $(w_1 - w_0)$ and the constant (K_2) to (fa/v), when the units of measurement are identical. If we thus write that $K_2 = fa/v$, then we

have that $K_2v = fa$. As (K_2) defines the specific rate of absorption per unit of volume and (f) the corresponding rate per unit of area, the consistent dimensional relationship existing between these two constants is apparent.

As the common system of logarithms is more generally used in ordinary calculations, the constant (k_2) based on this system will be referred to more specifically in this paper. If the rate dD_t/dt be denoted by the symbol (r_t) and $2.3k_2$ be substituted for its equivalent K_2, then we may write:

$$r_t = -2.3k_2D_t \tag{4}$$

which gives this rate numerically in terms governed by the units of time (t) and of the oxygen deficiency (D_t) actually applied.

Example:

Assume:

$D_t = 4$ p.p.m.
$k_2 = 0.2$ ("t" in days)
$r_t = 2.3 \times 0.2 \times 4 = 1.84$ p.p.m. per day

The value of (K_2) or (k_2), which defines the specific rate of reaeration, was found by Streeter and Phelps [2] to be influenced in the Ohio River by velocity of flow, mean depth, slope and roughness of the channel, and total volume of diluting water, which is measured by the stream discharge. These relationships were summed up in the expressions:

$$k_2 = \frac{cV^n}{H^2} \tag{5}$$

or

$$k_2 = \frac{c'V^n}{Q}$$

in which (V) is the velocity of flow, (H) the mean depth and (Q) the discharge, (c), (n), (c') and (n') being empirical constants depending on physical channel conditions and the other terms are as above defined.

The value of (K_2) or (k_2) also appears to be influenced to some extent by variations in the stream temperature, which affect the rate of oxygen absorption at the surface film and likewise its rate of diffusion through this film. From the results of some gas-absorption tests reported by Haslam, Hersey and Keen,[3] an empirical formula for this temperature effect was proposed several years ago by Streeter,[4] as follows:

$$k_2(T^\circ \text{ C.}) = k_2(20^\circ \text{ C.})[1.0159^{(T-20)}] \tag{7}$$

In this connection, it has been noted by Gould [4] that if the value of (k_2) is calculated from dissolved oxygen increases expressed in terms of percentage saturation, this temperature relationship may be practically

disregarded, as in such an event the decrease in oxygen saturation value with increased temperature tends to offset the effect on (k_2) of a corresponding increase in the rate of reaeration. Further observations are needed, however, to confirm these data under stream-flow conditions.

When the value of (k_2) is known, the rate of reaeration corresponding to a given degree of oxygen saturation deficiency may be calculated directly, as previously noted, from equation (4), having due regard for the units of time (t) and oxygen deficiency (D_t) actually followed. Thus, if (t) be expressed in days and (D_t) in milligrams per liter or parts per million, then the rate calculated from (4) will be given in terms of milligrams per liter of water per day. If the total volume of water is known, this rate can be readily transformed to terms of pounds or kilograms of oxygen absorbed per day by a given stretch of a stream exposed to reaeration. If, for example, (Q) is the discharge of the stream and (t) the time of flow in days between two given points, the total volume of water in this stretch will be $86{,}400Qt$ cubic feet. As one mg. of oxygen per liter of water is equivalent to 0.0000625 lb. per cubic foot, then from equation (4) the value of (r_t) in pounds per day will be:

$$r_t = (2.3k_2)(0.0000625D_t)(86{,}400Qt)$$
$$= 12.42k_2D_tQt \text{ lbs. per day} \qquad (8)$$

Example:

Assume:

$D_t = 4$ p.p.m.
$k_2 = 0.2$
$Q = 1000$ cu. ft. per sec.
$t = 2$ days.
$r_t = 12.42 \times 0.2 \times 4 \times 1000 \times 2 = 19870$ lbs. per day

If the oxygen deficiency (D_t) be reduced initially to terms of pounds per day,* then equation (8) may be rewritten as:

$$r_t = 2.3k_2D_tt \text{ lbs. per day} \qquad (9)$$

which is the same as equation (4) except that it contains the time of flow (t) between the two points.

Example:

Assume:

$D_t = 5.4 \times 1000 \times 4 = 21600$ lbs. per day
$k_2 = 0.2$
$t = 2$ days
$r_t = 2.3 \times 0.2 \times 21600 \times 2 = 19870$ lbs. per day

* Lbs. per day = 5.4 × Q (c.f.s.) × D_t (p.p.m.).

Where reaeration follows the unmodified absorption curve, such as is represented by equation (3*a*) and (3*b*), the total amount of reaeration (R_t) occurring up to any time (t) may be calculated by noting that $R_t = D_a - D_t$; whence

$$R_t = D_a - D_a \cdot 10^{-k_2 t} = D_a(1-10^{-k_2 t}) \tag{10}$$

This equation is analogous to the Theriault-Phelps formula:

$$X_t = L_a(1-10^{-kt}),$$

where (X_t) is the total amount of B.O.D. satisfied up to a given time (t).

Example:

Assume:

$$D_a = 6 \text{ p.p.m.}$$
$$k_2 = 0.2$$
$$t = 2 \text{ days}$$
$$R_t = 6(1 - 10^{-0.2\times 2}) = 6(1 - .398) = 3.7 \text{ p.p.m.}$$

Assume:

$$L_a = 6 \text{ p.p.m.}$$
$$k_1 = 0.1$$
$$t = 4 \text{ days}$$
$$L_t = 6(1 - 10^{-0.1\times 4}) = 6(1 - .398) = 3.7 \text{ p.p.m.}$$

Under natural stream conditions, in which biochemical oxidation of polluting matter results in a continuous withdrawal of dissolved oxygen from the stream water, an unmodified curve of oxygen recovery such as that defined by equation (10) is seldom if ever observed. In some instances, however, the rates of deoxygenation and of reaeration may tend to balance each other to such an extent that the oxygen saturation deficiency may remain very nearly constant in a considerable stretch of a river. Where this situation prevails, equation (8) or (9) may be utilized to calculate the average rate of reaeration when (k_2) is known, or vice versa, substituting for (D_t) the mean observed oxygen deficiency, which may be denoted as (D_m), and for (r_t) the corresponding average rate of reaeration, which may be denoted as (r_m), to distinguish it from the momentary rate (r_t).

One method of determining the value of (r_m) independently of (k_2) may be suggested from the data presented in the first section of this paper. If the total oxygen demand exerted on the stream between two points be denoted as (X_t), then the total amount of reaeration (r_m) may be calculated as the algebraic sum of (X_t) and any observed change in the dissolved oxygen content of the stream water between these two points (corrected for the effect of intermediate inflow). Thus, if (O_A) and (O_B) represent the dissolved oxygen content at two stations A and B, the

former upstream from the latter, then (r_m) may be calculated from the simple relation:

$$r_m = X_t + (O_B - O_A) \tag{11}$$

or

$$r_m = X_t + (D_A - D_B) \tag{12}$$

where (D_A) and (D_B) are the oxygen saturation deficiencies corresponding to (O_A) and (O_B).

Example:

Assume: Mean Water Temperature = 20° C.

$X_t = 8$ p.p.m.
$O_A = 4$ p.p.m. $(D_A = 9.2 - 4.0 = 5.2$ p.p.m.)
$O_B = 6$ p.p.m. $(D_B = 9.2 - 6.0 = 3.2$ p.p.m.)

$$r_m = 8 + (6 - 4) = 10 \text{ p.p.m.} \tag{11}$$
$$= 8 + (5.2 - 3.2) = 10 \text{ p.p.m.} \tag{12}$$

If the discharge $Q = 1000$ c.f.s., then $r_m = 5.4 \times 10 \times 1000 = 54000$ lbs. per day.

Where the conditions of observation are such that the value of (X_t) is approximated very nearly by the observed decrease in total oxygen demand of the stream water between two points,* then, if (L_A) and (L_B) denote the B.O.D. at points A and B, the quantity $(L_A - L_B)$ may be substituted for (X_t) and equation (11) rewritten thus:

$$r_m = (L_A - L_B) + (O_B - O_A) \tag{13a}$$

or

$$r_m = (O_B - L_B) - (O_A - L_A) \tag{13b}$$

Example:

Assume:

$L_A = 15$ p.p.m.
$L_B = 7$ p.p.m.
$O_A = 4$ p.p.m.
$O_B = 6$ p.p.m.

$$r_m = (15 - 7) + (6 - 4) = 10 \text{ p.p.m.} \tag{13a}$$
$$= (6 - 7) - (4 - 15) = 10 \text{ p.p.m.} \tag{13b}$$

In connection with its report on the disposal of Chicago sewage, the Sanitary District's Engineering Board of Review gave the term "oxygen balance" to the quantities $(O_A - L_A)$ and $(O_B - L_B)$ and calculated the net reaeration between two points as the increase in "oxygen balance," which is given by equation (13*b*). This method gives accurate results only when the difference $(L_A - L_B)$ affords a true measure of the total oxygen demand exerted on the stream between two points, as indicated above.

* See Part I, THIS JOURNAL, 7, 266–268 (1935).

The Resultant Oxygen Curve

The combined influence of deoxygenation and reaeration on progressive changes in the dissolved oxygen content of a polluted stream are well known and frequently observed below major sources of pollution. The time-curve traced by these changes in D.O. content is usually a very characteristic one, readily observable in a stream, unless it be disturbed unduly by intermediate inflow. Its general form is that of an inverted bow, with an extended limb reversing its curvature in a downstream direction and approaching an asymptote represented by complete oxygen saturation. It is often termed as the "oxygen sag" curve, because of its tendency to sag in the "inverted bow" section, with a minimum point marking the critical zone of oxygen depression in the stream resulting from the initial stage of rapid deoxygenation immediately below the source of pollution.

In connection with a study of oxidation and reaeration in the Ohio River, Streeter and Phelps [2] developed this "oxygen sag" curve mathematically, basing their formulation on two simple assumptions, both in accord with known laws governing the phenomena considered. The first assumption was that the rate of deoxygenation of a stream is directly proportional at any time to the unsatisfied oxygen demand of the stream at that time. The second was that the rate of atmospheric reaeration (the only form considered) is directly proportional at any time to the oxygen saturation deficiency existing in the stream at that time. As previously noted in this paper, both of these assumptions appear to be in accord with well-established observational fact.

From these two assumptions, a simple first-order linear differential equation was set up, defining the rate of change in the oxygen saturation deficiency (D_t) at any time as the resultant of two opposing rates, one (i.e. of deoxygenation) tending to depress the D.O. content and the other (of reaeration) to increase it up to the saturation point at the prevailing stream temperature. This differential equation may be expressed as follows, in terms similar to those previously used in this paper:

$$\frac{dD_t}{dt} = K_1 L_t - K_2 D_t$$

where D_t = the dissolved oxygen saturation deficiency at any time (t),
L_t = the remaining unsatisfied oxygen demand at the same time,
K_1 = a constant defining the specific rate of deoxygenation, and
K_2 = a similar constant defining the specific rate of reaeration.

The following integrated form of this equation gives the value of (D_t) at any time:

$$D_t = \frac{K_1 L_a}{K_2 - K_1} (e^{-K_1 t} - e^{-K_2 t}) + D_a \cdot e^{-K_2 t} \qquad (14)$$

where L_a = the oxygen demand at an initial reference point (A) in the stream,

D_a = the dissolved oxygen saturation deficiency at the same point,

and e = the Naperian logarithmic base (= 2.7183 approximately).

When the common logarithmic base (10) is substituted for the Naperian base (e), in order to facilitate computation, this equation may be written:

$$D_t = \frac{k_1 L_a}{k_2 - k_1}(10^{-k_1 t} - 10^{-k_2 t}) + D_a \cdot 10^{-k_2 t} \qquad (15)$$

where $k_1 = 0.434K_1$ and $k_2 = 0.434K_2$

As the constants (k_1) and (k_2) are directly comparable numerically with those referred to previously in this paper, equation (15) will be used more specifically in connection with the text which follows. Both equations (14) and (15) will give identical results if the proper constant ratio of (K_1) and (K_2) to (k_1) and (k_2) is consistently observed.

In connection with their derivation of these two formulae, Streeter and Phelps [2] showed that the minimum value of (D_t), marking the lowest point of the "oxygen sag" curve, may be derived from the expression:

$$e^{(K_2 - K_1)t} = \frac{K_2}{K_1}\left[1 - \frac{D_a(K_2 - K_1)}{K_1 L_a}\right]$$

which was obtained by differentiating equation (14) with respect to time and placing the resulting expression equal to zero. If this expression be transformed to terms of the constants (k_1) and (k_2), it may be written thus:

$$10^{(k_2 - k_1)t} = \frac{k_2}{k_1}\left[1 - \frac{D_a(k_2 - k_1)}{k_1 L_a}\right]$$

whence

$$t = \frac{\log\left\{\frac{k_2}{k_1}\left[1 - \frac{D_a(k_2 - k_1)}{k_1 L_a}\right]\right\}}{(k_2 - k_1)} \qquad (16)$$

Equation (16) gives the time (t) corresponding to the minimum point of the oxygen "sag" curve, using common logarithms. By calculating this and substituting it into equation (15), the ordinate (D_t) of the curve at the minimum point can be readily computed. This minimum point marks the approximate center of the critical zone of oxygen depletion in a stream below a major source of pollution. Its determination is often a matter of practical importance where maintenance of a specified minimum dissolved oxygen content in a stream is required.

Application of Formulae

The formulae described in the foregoing text are applicable to a number of practical problems which arise in connection with stream

sanitation. For purposes of illustration, it will suffice here to discuss only two of these problems, leaving until some future time a further development of this phase of the subject.

The first problem considered will be that of establishing proper values of the coefficients (k_1) and (k_2) for a particular stream, under various conditions of temperature, flow and pollution such as are likely to be significant in estimating the capacity of that stream for natural oxidation.

The coefficient (k_1) is a measure of the composite rate of deoxygenation between two given points in a stream; hence it should be based, as nearly as possible, on the actual total amount of oxygen demand exerted on the stream between these points, including that of any sludge deposit. In the first part of this paper this total exerted oxygen demand has been designated by the symbol (X_t) and methods of calculating it have been described. Using this same symbol and denoting the observed B.O.D. at the lower of the two observation points, corrected for inflow, as (L_B), we then may note that the quantity (L_a) in equation (15), representing the initial unoxidized B.O.D., should be equivalent to ($L_B + X_t$) and that

$$\log \frac{L_B}{L_a} = \log \frac{L_B}{(L_B + X_t)} = - k_1 t$$

whence

$$k_1 = - \frac{1}{t} \log \frac{L_B}{L_B + X_t} \tag{17}$$

Where conditions are such that the total oxygen demand exerted by the stream between two points is measured directly by the observed decrease in the total B.O.D. of the stream water between these points, (See Part I of this paper), the observed B.O.D. at the upper station, which may be denoted as (L_A), may be substituted for ($L_B + X_t$) in equation (12), which then would read:

$$k_1 = - \frac{1}{t} \log \frac{L_B}{L_A} \tag{18}$$

In this as in the above-noted case, (L_B) would be previously corrected for the effect of intermediate inflow.

The determination of the reaeration coefficient (k_2) may be made by two different methods. The first method, an approximate one based on averages of direct observations, involves the use of equation (8) or (9), having previously determined the average rate of reaeration (r_m) by direct calculation according to equation (11) or (12) or by means of equation (13) if conditions justify its application. Assuming that the discharge (Q) and time of flow (t) are also known from measurement, and from the former being able to express (r_t) and (D_t) in terms of pounds of oxygen absorbed per day, we then may calculate the unknown (k_2) by

the following simple transposition of equation (9), with (r_m) and (D_m) substituted for (r_t) and (D_t):

$$k_2 = \frac{r_m}{2.3 D_m t} \tag{19}$$

If (D_m) and (r_m) are expressed in parts per million, then applying equation (4), we may write:

$$k_2 = \frac{r_m}{2.3 D_m} \tag{20}$$

A second method of calculating (k_2), as originally described by Streeter and Phelps,[2] involves solving for it by indirect substitution in equation (15). In this case trial values of (k_2) are assumed and corresponding values of (D_t) are calculated, (L_a), (D_a) and (k_1) being known from actual measurements. From a plot of the assumed values of (k_2) against the calculated values of (D_t), a value of (k_2) corresponding to the observed oxygen deficiency (D_B) at the lower reference point (corrected for inflow) is taken. This value should be a very close approximation to the true value of (k_2) under the conditions embraced by the observations. Ordinarily, not more than three or four trial calculations are necessary to fix the curve from which the proper value of (k_2) may be derived. In making this calculation, (L_a), (D_a) and (D_t) may be expressed either in parts per million or in pounds per day, though the particular unit followed should be the same, of course, for all three of these quantities.

In order to illustrate the application of these two methods, the following example will be given, with the same data as were utilized in the example cited in connection with the first part of this paper, in which the total oxygen demand exerted by the Illinois River in a stretch extending from Joliet to Peru, a river distance of about 46 miles, was calculated from yearly averages of data collected by the U. S. Public Health Service in 1921–22. In the example which follows, this calculated total demand figure will be part of the base analytical data, which will be expressed, as before, in terms of thousands of pounds of oxygen per day (M#/dy).

The symbols to be used in the calculation are the following:

L_A = the observed total first-stage B.O.D. at the upper station, in thousand pounds per day (M#/dy).

L_B = the corresponding observed B.O.D. at the lower station, corrected for inflow (M#/dy).

D_A = observed dissolved oxygen saturation deficiency at the upper Station A (M#/dy).

D_B = corresponding observed saturation deficiency at the lower station B, corrected for inflow.

D_t = calculated oxygen saturation deficiency at the lower station, corresponding to a known time of flow (t).

D_m = mean observed oxygen saturation deficiency in the river stretch extending from Station A to Station B $\left(= \frac{D_A + D_B}{2} \right)$.

X_t = total B.O.D. exerted (M#/dy) on the stream between the two stations.

k_1 = coefficient of deoxygenation.

k_2 = coefficient of reaeration.

t = time of flow between the two stations, in days.

T = stream temperature, in °C.

L_a = initial B.O.D. at Station A, calculated from (L_B) and (X_t), (M#/dy).

Q = stream discharge in cu. ft. per second, corrected for inflow between Stations A and B.

r_m = observed average net rate of reaeration (M#/dy), between Stations A and B.

Base Data

L_A = 1144	M#/dy	D_A = 275	M#/dy	T = 11.9°	C. (Avge.)
L_B = 854	"	D_B = 271	"	Q = 9190	c.f.s.
X_t = 288	"	D_m = 273	"	t = 1.05	days

Calculations

Method 1—By Direct Substitution into Equation (19)

Basic equation:

$$k_2 = \frac{r_m}{2.3 D_m t} \tag{19}$$

$r_m = X_t + (D_A - D_B) = 288 + (275 - 271) = 292$ M#/dy.

$D_m = 273$ M#/dy.

$t = 1.05$ days.

$$k_2 = \frac{292}{2.3 \times 273 \times 1.05} = 0.44.$$

Method 2—By Indirect Substitution into Equation (15)

Basic equation:

$$D_t = \frac{k_1 L_a}{k_2 - k_1} (10^{-k_1 t} - 10^{-k_2 t}) - D_a \cdot 10^{-k_2 t} \tag{15}$$

$D_a = D_A = 275$ M#/dy.

$L_a = L_B + X_t = 854 + 288 = 1142$ M#/dy.

$$k_1 = \frac{1}{1.05} (\log 854/1142) = 0.120.$$

$k_1 L_a = 1142 \times 0.120 = 137.$

Try $k_2 =$	.30	.40	.45
$k_1 =$	.12	.12	.12
$k_2 - k_1 =$	.18	.28	.33
$\frac{k_1 L_a}{k_2 - k_1} =$	761	490	415
$10^{-k_1 t} =$	.748	.748	.748
$10^{-k_2 t} =$	.484 | × 275 = 133	.380 | × 275 = 104	.337 | × 275 = 92
Diff. =	.264 | × 761 = 201	.368 | × 490 = 180	.411 | × 415 = 170
$D_t =$	334	284	262

As the correct value of (k_2) would correspond to the observed value of D_B = 271 M#/dy, this value may be obtained by plotting the trial values of (k_2) against the calculated values of (D_t). From such a plot, k_2 = 0.43 (See Figure I).

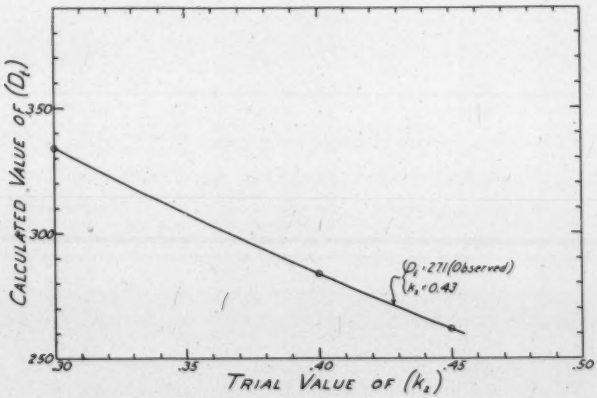

FIG. I.—Plot of trial values of (k_2) against calculated values of (D_t), with indicated value of (k_2) corresponding to observed value of (D_t).

The divergence between the values of (k_2) obtained in this case by the two methods of calculation is so small as to be hardly significant, though in other instances it might be considerable. In general, Method 1 will give a more accurate result where the dissolved oxygen content of the stream either remains nearly constant or varies uniformly in a given stretch, so that it may be fairly represented by an average value based on terminal observations. Method 2 will yield the better result where the

dissolved oxygen content tends to follow more closely the true "oxygen sag" curve between two given observation points.

Where practicable, it is desirable to determine values of (k_2) in a given stream under varying conditions of flow, in order to establish the range of these values, for different flows and if possible, the relationship existing between (k_2), stream discharge and velocity of flow, such as indicated, for example by equation (6). Otherwise, a very rough correction of (k_2) for variations in flow may be applied by assuming that (k_2) varies directly as the velocity of flow and inversely as the discharge.

When once determined for a particular river stretch, values of (k_2) should be characteristic of any given flow condition, regardless of the oxygen demand imposed on the stream, and hence may be used for calculating the probable trend of the oxygen "sag" curve for any assumed degree of pollution in the stream. In this connection, the problem frequently arises of ascertaining the maximum permissible pollution of a stream at a certain point which, under a given flow condition (usually assumed as the most unfavorable), may be consistent with maintaining a dissolved oxygen content not less than a specified minimum designed to protect normal aquatic life and prevent "nuisance." The following example will illustrate the method of calculation followed in this case.

Let us assume a stream having a summer discharge of 1000 cu. ft. per second, having a mean summer temperature of 24° C., with a dissolved oxygen content of 6 p.p.m., and a total first-stage B.O.D. of 5 p.p.m. at a point immediately above a city. Let it be further assumed that the rate of reaeration of the stream is such that the value of (k_2) is 0.20 for the given flow condition and that the total rate of deoxygenation is such that the value of (k_1) is 0.12 at the mean temperature of 24° C. The problem is to determine the maximum B.O.D. which can be imposed on the stream by the combined wastes of the city, consistent with maintaining a minimum dissolved oxygen content not less than 3 p.p.m. under the summer condition assumed.

In this case, there are two unknown quantities, one being the time (t) to the point of minimum D.O. content and the other the maximum value of (L_a) consistent with the condition stated. The known quantities are: $k_1 = 0.12$, $k_2 = 0.20$, $D_a = [8.4\frac{1}{2} - [6.0 = 2.4$ p.p.m. (8.4 being the D.O. saturation value at 25° C.), and $D_t = 8.4 - 3.0 = 5.4$ p.p.m. at the minimum D.O. point.

The first step is to assume a trial value of the total initial B.O.D. (L_a) in equation (16) and to calculate from this equation the corresponding time (t) to the point of minimum D.O. content. Let us assume that $L_a = 15$ p.p.m. Then substituting the known terms into equation (16),

we have

$$t = \frac{\log\left\{\frac{k_2}{k_1}\left[1 - \frac{D_a(k_2 - k_1)}{k_1 L_a}\right]\right\}}{k_2 - k_1} \qquad (16)$$

$$= \frac{\log\left\{\frac{.20}{.12}\left[1 - \frac{2.4(.20 - .12)}{.12 \times 15}\right]\right\}}{.20 - .12}$$

$$= \frac{\log[1.67(1 - .107)]}{.08}$$

$$= \frac{\log 1.49}{.08} = \frac{.173}{.08} = 2.16 \text{ days.}$$

By substituting this value of (t) into equation (15) and noting that $D_t = 5.4$ p.p.m., we then may obtain a first approximation of the proper value of (L_a).

Thus:

$$D_t = \frac{k_1 L_a}{k_2 - k_1}(10^{-k_1 t} - 10^{-k_2 t}) + D_a \cdot 10^{-k_2 t} \qquad (15)$$

$$5.4 = \frac{.12 L_a}{.08}(10^{-.12 \times 2.16} - 10^{-.20 \times 2.16}) + 2.4 \times 10^{-.20 \times 2.16}$$

$$= .15 L_a \times .179 + 2.4 \times .371$$

$$= .268 L_a + .890.$$

$$.268 L_a = 5.4 - .89 = 4.51$$

$$L_a = 16.8 \text{ p.p.m.}$$

If this value of (L_a) now be substituted back into equation (16), instead of the trial value of 15 p.p.m. originally assumed, a readjusted value of $t = 2.24$ days is obtained as the time to the minimum D.O. point. Substituting this value into equation (15), a second approximation of $L_a = 16.6$ p.p.m. is obtained. As this value deviates from the first approximation by only 0.2 p.p.m., it is sufficiently close for all practical purposes.

As it was originally assumed that the stream immediately above the city contains 5 p.p.m. B.O.D., the net amount of B.O.D. which can be imposed on the stream by the combined wastes of the city will be $16.6 - 5.0 = 11.6$ p.p.m. With a discharge of 1000 second-feet, this will be equivalent to $5.4 \times 1000 \times 11.6 = 6260$ lbs. per day of total B.O.D. Assuming 0.25 lbs. of B.O.D. as the normal per capita contribution as domestic sewage, this will be equivalent to the domestic sewage of roughly 25,000 people. Comparison of this figure with the actual sewered population of the city plus the population-equivalent of the total industrial wastes contribution, would give a fair measure of the requirements to be met by sewage and industrial waste treatment, if this total population were in excess of the permissible maximum.

In some cases the problem presented may be that of estimating the minimum allowable stream flow required, with a given degree of pollution, to maintain a specified minimum dissolved oxygen content, or excess of dissolved oxygen over oxygen demand, in a stream. In cases of this kind, the method of solution is similar to that above described, except that the total oxygen demand contributed to the stream is fixed initially and the solution of the problem consists in determining the minimum allowable discharge consistent with a maximum value of (L_a) sufficient to maintain the desired minimum of D.O. content in the stream.

Although it is necessary, in applying equations (15) and (16), to express the rate of reaeration in terms of the coefficient (k_2), it is often more convenient to record this rate in surface units, such as in grams per square meter or pounds per thousand square feet, of water surface per day. Where this latter method of expression is followed, it is essential to bear in mind that the rate thus expressed will vary in a given body of water with the degree of oxygen saturation, which always should be recorded with a particular rate observed. In a natural stream, the observed surface rate of reaeration will seldom, if ever, vary in direct proportion to the oxygen saturation deficiency, as variations in the latter are usually coincident with changes in flow conditions, which in themselves affect the rate of reaeration. An example is afforded by the results of Mahr's [2] observations in several German rivers, which he has recorded in a diagram reproduced in Figure 2. In this diagram it will

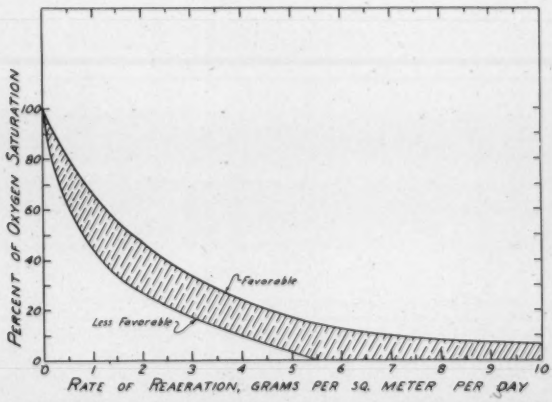

FIG. II.—Plot, after Mahr, showing observed rates of reaeration corresponding to different percentages of dissolved oxygen saturation in several German rivers. (Mahr, *The Surveyor*, **78**, No. 2030, pp. 618–619, Dec. 19, 1930.)

be noted that Mahr has drawn two limiting curves showing the surface rates of reaeration observed concurrently with varying degrees of oxygen saturation, one curve representing conditions favorable and the other, conditions less favorable to reaeration. Although it is not clear in this case whether full allowance has been made for the effects of biochemical deoxygenation in calculating the rates of reaeration shown, it seems likely that the departure of the curves from a straight-line trend probably reflects to a considerable extent a gradual change in flow conditions with increasing or diminishing degrees of oxygen saturation. In a later paper it will be shown that when these variable factors can be held constant, surface rates of reaeration observed under stream-flow conditions have shown a marked tendency to bear a straight-line relation to the oxygen saturation deficiency in the underlying stream.

Although surface rates of reaeration tend to remain more nearly constant for a given stream under variable flow conditions than do values of the reaeration coefficient (k_2), values of the latter derived from observations in different streams have shown a rather striking degree of mutual consistency, when obtained under fairly similar flow conditions, particularly those of summer low-water flow. In this connection it is of interest to mention values of 0.22, 0.24 and 0.21 in three stretches of the Ohio River and of 0.27 and 0.27 in two stretches of the Illinois River, both under summer conditions of May–September, as noted elsewhere by Streeter;[4] values ranging from 0.12 to 0.27 in the Connecticut River under summer conditions, as noted by Setter;[5] and values of 0.26 and 0.27 in a small impounded stream near Morris Plains, N. J., under summer and winter conditions, respectively, as noted by Ridenour.[6]

Where hydraulic dimensions, particularly those affecting the velocity of flow and the ratios of water surface area to depth and volume of flow, are fairly similar for different streams, values of the reaeration coefficient (k_2) observed in such streams should be of about the same order of magnitude, as above indicated. On the other hand, the wide variation in these values obtained by Streeter and Phelps[2] in the same stretches of the Ohio River under divergent flow conditions indicates the marked extent to which they can be influenced by these conditions.

Concluding Summary

The present paper, consisting of two sections, has been prepared with the object in view of setting down a concise but reasonably comprehensive statement of that which is known at this writing concerning the nature, underlying principles and practical measurement of natural oxidation in polluted bodies of water, taking flowing streams as a particular example. In this connection, methods and formulae have been described whereby rates of oxidation and reaeration and the trend of

the resultant oxygen curve may be calculated from observations of progressive changes in the biochemical oxygen demand and dissolved oxygen content of a polluted stream water, together with certain collateral data bearing on the volumes and times of stream flow between various observational points. The constants of these formulae have been discussed and several gaps in existing knowledge of them have been noted.

In a very large number of cases, possibly constituting a majority, the measurement of rates of oxidation may be greatly simplified by basing it on the observed decrease in the total biochemical oxygen demand of the stream water between two given points, after correcting this decrease for the effect of intermediate inflow. In these cases, the observational data should cover a period sufficiently long and stream conditions during this period should be sufficiently well-balanced to allow a reasonably stable equilibrium to be reached between the rate of deposition, if any, of oxidizable matter in the bottom sludge deposit, and the rate at which this deposit is being oxidized. From graphic examples shown in connection with the first section of this paper, the length of such a period would depend on the specific rate of oxidation in the bottom deposit and this, in turn, probably depends very considerably on the stream temperature, though further data are needed as to the nature and extent of this temperature relationship. Under summer conditions this equilibrium period should not exceed thirty days or less. Under winter conditions it might be somewhat longer.

Where conditions are favorable to utilizing the simplified method of calculation above indicated, the corrected observed rate of decrease in the biochemical oxygen demand of the stream water may also be taken as the basis of calculating the rate of reaeration. As this latter rate is characteristic for a given physical stream condition, it can be applied to further calculation of the resultant oxygen curve, with its probable minimum point, for any assumed initial condition of pollution expressed in terms of biochemical oxygen demand. For these several calculations, formulae given in connection with the text may be utilized, in accordance with methods which have been described.

In river stretches in which oxidation is proceeding wholly within the first or carbonaceous stage, the total first-stage biochemical oxygen demand may be considered as the total oxygen demand of the stream water for purposes of calculation. Wherever oxidation passes definitely from the first to the second stage of oxidation between two given points in a stream, this transition should be taken into account in calculating the total change in oxygen demand between these points, following a method similar to that described in the first section of this paper.

At the present writing, further information is needed concerning: (1) the rates at which bottom sludge deposits of various depths and rela-

tive ages are oxidized in streams at different temperatures; (2) the degree to which readily oxidizable "enzymic" products of bacterial decomposition, connected with the "immediate" oxygen demand, are liberated from these deposits under various conditions and the effects of temperature changes on their subsequent rate of oxidation in the stream, and (3) the extent to which rates of atmospheric reaeration under natural stream flow conditions are affected by variations in temperature, stream depth, velocity of flow and roughness of the channel. These questions are now being studied at the U. S. Public Health Service's Stream Pollution Investigations Station at Cincinnati, making use of experimental stream-flow channels. It is expected that the results of these studies will be published from time to time as they may be completed.

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A NATIONAL PROGRAM FOR WATER POLLUTION CONTROL. CONSIDERATION OF SOME ELEMENTS IN ITS FORMULATION AND ADMINISTRATION *

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There has been an increasing appreciation by the people of this country of the necessity for a consistent reduction in the extent to which our surface waters are being degraded by contributions of domestic and industrial sewage. This attitude is finding more and more frequent expression in the actions of representative bodies of citizens such as that of the American Association for the Advancement of Science with more than 18,000 members and 141 associated societies and academies representing a total membership of more than 725,000 which at its recent meeting in Pittsburgh adopted the following resolution concerning the pollution of public waters:

"Whereas the degree of control of pollution of coastal and inland waters by domestic and industrial wastes, which is essential for public health and national welfare, has not yet been achieved and can not be without further development and coordination of Federal, state and local authority, *Therefore, be it resolved* that the American Association for the Advancement of Science, meeting at Pittsburgh, urgently recommends the adoption of legislation adequate to control pollution of public waters."

There are several outstanding causes for this growing demand for the sanitary protection of our water resources among which may be mentioned: (*a*) the necessity for conserving the quality of our sources of public water supply, (*b*) increasing nuisance and damage suffered by riparian owners, (*c*) deterioration in recreational and aesthetic value of streams, lakes and coastal waters affecting a large percentage of our population, and (*d*) economic losses imposed upon the industrial and commercial uses of surface waters including those of fish and shell fish production. Each one of us, no doubt, could cite specific instances coming under his personal observation in which one or more of these objectionable conditions has forced upon him the conclusion that some adequate relief must sooner or later be provided.

The increasing urge, or even demand, for improvement in this situation can be relieved and counteracted permanently only by the estab-

* Presented before the New Jersey Sewage Works Association, Trenton, N. J., March 21, 1935.

lishment and support of adequate control authority, comprehensive planning of water pollution abatement measures and reasonable but consistent enforcement of stream cleaning policies on a national scale. This is admittedly a far-reaching activity but the exigencies of the case require broad corrective measures if these problems are to be solved conclusively and in the interest of the greatest good to the greatest number. The membership of sewage works organizations includes the type of trained men upon whom a large part of the responsibility for carrying out such a program must be placed. We should be prepared to assume this responsibility not only in the execution of the plan but in the assumption of intelligent leadership during its formulation to insure that only reasonable and practicable procedures are incorporated. To stimulate thought and focus attention on these matters, is the object of this paper.

The Extent of the Water Pollution Problem

Before discussing remedial suggestions, some general observations concerning the extent and intensity of water pollution may be helpful. It is reasonable to suppose that urban population is, in general, a reasonable index of stream pollution by domestic sewage. As of 1930, there were 3165 incorporated places in the continental United States with a total of 68,955,000 population, or 56 per cent of the entire population of the country. This urban population is not, however, equally distributed over the entire United States. In the section north of the Ohio and Potomac and east of the Mississippi Rivers, comprising 14 per cent of the total area, are located 1635 or 51.7 per cent of the total number of towns, containing 65.4 per cent of the total urban population of the country. Moreover, in this same north-eastern section, the rural population, which presumably suffers some harmful effects resulting from urban and industrial pollution, averages 40.5 per square mile in comparison with 14.1 per square mile for the remainder of the country. Altogether, the combined urban and rural population in this area comprises 50.6 per cent of that of the continental United States. It is to be expected, therefore, that in general—while there are numerous exceptions—water pollution and damage suffered therefrom would be most intensive in this north-eastern division, a section where recreational facilities are also highly developed.

The intensity of water pollution may be gaged in general terms by the amount of domestic sewage and industrial wastes contributed, making proper allowance for the proportion of these wastes which receives artificial treatment before discharge to the receiving waters. For the country as a whole, comprehensive data on sewered populations are nonexistent. On the entire Ohio River watershed of some 203,000 square

miles, a complete survey in 1914–1916 by the Public Health Service, revealed that 72.1 per cent of the total urban population was connected to sewers. A similar survey of the Illinois watershed in 1920–1922 showed that, exclusive of Chicago, 76 per cent of the urban population was served by sewers, while on the upper Mississippi River, exclusive of the Minneapolis-St. Paul Metropolitan District, this figure was found to be 68 per cent. Based on these data, an estimate of 75 per cent of the urban population connected to the sewers would appear reasonable and conservative.

The relative polluting effect of industrial wastes is even more difficult to estimate and will vary widely in different areas depending upon the nature of the dominant manufactured products. Considering organic pollution only, on the Ohio River watershed, our survey showed that the industrial wastes were equivalent to 124 per cent of that contributed by domestic sewage. On the Illinois River the population equivalent of industrial wastes was 85 per cent and on the upper Mississippi River 92 per cent. It would probably be fair to say that, generally, the organic pollution contributed by industry is about equal to that contributed by population.

Combining these two estimates would result in the conclusion that the total organic pollution load is equivalent to that of the domestic sewage of 1½ times the total urban population of the country. This potential load of pollution would be reduced, however, in proportion to the extent of treatment effected before these wastes are discharged to the receiving waters. Again, the extent and efficiency of sewage treatment processes vary widely in different parts of the country. It can be said safely that the volume of sewage being treated is increasing much more rapidly than is the population contributing sewage and also that the general, overall efficiency of sewage purification has improved. However, there are few reliable data available as to the proportion of the total raw sewage of this country that is being treated. Referring again to the Ohio River survey—at that time 11.8 per cent of the sewered population were discharging sewage through some form of treatment process. The Conference of State Sanitary Engineers Committee finds that in 33 states reporting, about 50 per cent of all sewered communities have some form of treatment for at least a part of their sewage. These treatment processes range from screening and sedimentation to oxidation and chlorination. On the other hand, at the time of our surveys (1920–1927) no appreciable quantity of sewage was being treated on the Illinois River watershed and only 8.3 per cent of the total on the Upper Mississippi. Taking into consideration the average efficiency of all treatment plants as a group, and considering that only a relatively small percentage of industrial wastes are treated at all, it is

likely that at present the potential organic sewage pollution load is not being reduced by treatment by more than 20 per cent at most. On this basis the equivalent of 1.2 times the urban population of the country or 82,700,000 persons contribute raw sewage directly to our surface waters. Applying this same estimate to the northeastern portion of the United States, the waters of this area are receiving the equivalent of untreated sewage from 54 millions of people or nearly half the total population of the country.

This excessive pollution of our public waters affects directly the health, comfort and economy of human life in varying degree in all sections of the country. Examples of such effects are: (1) unpalatable and hard public water supplies, (2) outbreaks of water-borne gastro-intestinal disorders and occasional epidemics of typhoid, (3) abandonment of many streams for recreational purposes, (4) widespread damage to fish life, including shell fish, (5) economic industrial loss through increased costs of water treatment for various trade purposes and steam power plants and reflected directly in limitation of the industrial development in such areas, (6) damage to agriculture. It is pertinent to note that such adverse effects create widespread problems that very frequently transcend local and state boundaries.

Planning Remedial Measures

The broad extent of this problem of surface water pollution calls for carefully considered plans if adequate and effective relief measures are to be instituted. Such measures are of two general types. The first is concerned with the determination of proper technical procedures and treatment processes for dealing most economically with polluting materials to insure adequate stream protection but at the same time not to impose degrees of waste treatment in excess of those essential for effecting the desired results. The other type of planning has to do with the creation, definition of authority and methods for maintenance of the regulatory body or bodies necessary to stimulate and insure co-operative and efficient functioning of federal, regional, state and local government in a permanent program for improving and then maintaining the sanitary quality of our public waters.

Determination of essential relief measures for excessively polluted waters and the selection of economical processes for the treatment of pollutional materials are primarily technical problems, for the solution of which knowledge obtained through past and continuing research and experience are required. Such factors as the pollution load and its measurement, the natural recovery powers of polluted waters in motion or at rest, the conditions that influence these powers, the nature of use to be made of the waters of the receiving streams, are all points, among

others, upon which definite information must be available and appraised for each particular case, if maximum return is to be obtained in situations where corrective measures are to be applied. This appraisal is likewise necessary for the selection of proper processes for the treatment of sewage and industrial wastes best fitted to local conditions. All of these factors obviously play an important role in the choice and in the application of plans to every case where stream improvement is to be undertaken. It is just as important not to over-treat or over-correct pollutional conditions, as it is to provide adequate treatment. This balance can only be attained by careful, objective planning, the organization and maintenance of which activity is one of the prime requisites for a successful program of stream cleaning.

In general—and there are few exceptions—it must be admitted that comprehensive and equitable control machinery has not been placed in operation in this country for relief of stream pollution on any nation-wide or even watershed scale. Such advances as have been made, generally have been of a local character. As a result, a change in viewpoint has developed that is insisting that some definite and inclusive action be taken by the Federal Government to bring about uniform improvement in the sanitary condition of our public waters. These demands are of one of two general types: One is that the Federal Government be granted broad, inclusive and mandatory powers to take entire charge of all stream cleaning measures and be provided with sufficient funds to effect rapid relief wherever installation of such corrective procedures is indicated. The other general type of plan proposed is that the central government be authorized to assume directional responsibility for the development and execution of surface water sanitation procedures, working through and in co-operation with regional, state and other governmental agencies. As between these two suggested policies there is some conflicting opinion concerning the advantages of each.

The Community of Interest in Stream Sanitation.—There is a community of interest in the conservation of the quality of our public waters, and consideration of the extent of such interest should be helpful in arriving at some conclusions as to the most effective policy that will insure this conservation. These interests may be roughly grouped as local, state and federal. The local interest is more direct and confined to specific situations. It is concerned most generally with such particular problems as protection from excessive pollution of the source of the public or domestic water supply, the avoidance of local nuisance with resulting decline in real estate values and the train of other objectionable conditions that localized stream pollution entails, and the preservation of recreational facilities which clean waters provide. The state

interest is more inclusive and is concerned not only with the individual municipal problems, but with the preservation of the sanitary quality of intrastate streams and lakes as units. The state desires to conserve its internal water resources for all its people, including use for public water supply, industry and recreation, with particular emphasis on avoiding those objectionable features which affect the public health. The federal interest is still more comprehensive and from the broader viewpoint of the welfare of all its citizens. In this group would fall, among others, (1) preservation of the sanitary condition of interstate and navigable streams and lakes as well as of coastal waters and international and boundary waters, (2) the control of interstate spread of water-borne disease through such channels as unsafe drinking water and interstate exchange of improperly protected food and shell fish, (3) the conservation of surface waters for navigation and for prevention of flood damage on interstate watersheds. While these separate interests are necessarily overlapping in some details and are not always sharply defined, they do nevertheless, suggest a reasonable division of control responsibility that could be assumed for the solution of this water pollution problem.

The Allotment of Governmental Regulatory Power.—Bearing in mind the various interests concerned in water sanitation as enumerated above, logical successive divisions of territory for administrative authority and control purposes would seem to be: (1) Municipal or metropolitan areas, (2) States, (3) Regional or Interstate Watersheds and (4) National. The first two of these groups have been and are more or less active in the correction of the water pollution problems with which they are immediately confronted. The principal defect has been, perhaps, the lack of comprehensive planning of control procedures and the little stimulation and support contributed by the central government, in the establishment of definite objectives and practical, economical methods for attaining them. To supply this deficiency it would seem wise, therefore, not to discard all local and state organizations which have been functioning with more or less success, but to coordinate them as working parts in a more comprehensive and far-reaching arrangement in which they can be stimulated and supported in their particular spheres of action. For successive, superimposed steps in such a plan the watershed area authority and some suitable designated agency of the Federal Government appear advisable.

A regional or watershed administrative authority would seem to be essential for integration of policies and procedures where the correction of pollutional conditions of the public waters common to more than one state is undertaken. Only by some such device can the conflicting views and interests of individual states be composed and adjusted. Such an authority has not been developed or operated in this country for the

sanitary protection of surface waters. Some forward steps are in progress in this direction notably on the Ohio River and Great Lakes watersheds, mostly in the nature of voluntary joint action by individual state government departments. We can learn much concerning the usefulness, organization and operation of such river boards from the experience of England and Germany, where, in heavily industrialized and populated areas, they have offered the only practicable solution for intolerable conditions of excessive pollution of entire river systems.

In the same way, the organization and prosecution of a nation-wide program would seem to require the direction, stimulation and coordination of some authorized federal government agency which could function in these capacities as well as in that of the active representative of the larger federal interest as a whole. One existing difficulty of the individual state authorities has been the threat of organized industries to withdraw from a state where the fair enforcement of antistream pollution regulations is attempted. It is a fair question, indeed, as to whether industry has ever assumed the relative degree of responsibility for objectionable conditions of stream pollution which its contribution of pollutional wastes requires. If reasonable regulations were uniformly in force throughout the country, real obstacles of this nature would be largely eliminated. For the uniform enforcement and efficient conduct of any country-wide activity of this kind, the necessity for some central federal authority would seem to require no extended defense.

The extent of limitations of administrative and regulatory powers to be assigned to each of these four successive classes of governmental authorities requires most careful study. Obviously, such powers must be adjustable to local and regional conditions and to that extent quite flexible. In a general way they should conform to the particular functions of the class to which they apply, and at the same time permit, to the subordinate units, the utmost freedom of action consistent with the objectives of the larger program of sanitary protection of public waters. The decisions, actions and policies of these subordinate groups would of necessity have to be (for purposes of proper administration and coordination) subject to the review and approval of their ranking authority. Some observations concerning the distribution of these powers and responsibilities might not be out of place.

Responsibilities of Metropolitan Areas or Community Population Groups.—From these groups as a rule originate the major volumes of concentrated, polluting wastes, both domestic and industrial. Upon these groups, therefore, would appear to rest the prime responsibility of applying to their wastes, direct and adequate corrective measures prior to discharging them to receiving waters, the adequacy of such treatment to be determined by the higher state or watershed planning authority.

The detailed design, construction and operation of their sewage treatment plants would, therefore, be assigned to the individual communities, always subject to certification by the central authority, either state or regional, as to the need as well as to the sufficiency of the methods proposed. This procedure is practically now in effect in many states and has well demonstrated its usefulness.

Functions of States.—The duties and powers of state authorities require clear definition in that they are of a more or less dual nature. States should be responsible for the proper functioning of the direct treatment procedures of cities, metropolitan centers and industries, with mandatory powers to require enforcement of their edicts concerning pollution abatement. At the same time the state is the logical unit to develop comprehensive detailed plans for the protection of intrastate waters and sources of public water supplies within its borders. To carry on these functions adequately, continued observations and inspections of sewage treatment plants and receiving waters would be necessary, together with provision for research study of specific local problems as they arise. These functions too, are now assumed and are in operation by the health departments, water commissions or similar authorities in many states. However, these state authorities in turn should be so constituted that coordination and cooperation with the regional and federal organizations is assured.

Duties of Regional or Interstate Watershed Boards.—These regional groups, independent of state boundaries, are believed to be essential for the prosecution of stream cleaning programs comprising large interstate areas or coastal regions. They must obviously function in close cooperation with state authorities but should be vested with adequate powers to enforce their decrees uniformly as in their judgment is essential for the sanitation of river sysems and coastal waters as comprehensive units. They would, therefore, constitute the administrative agencies for stream cleaning activities in watershed areas within their respective jurisdiction. Such Board would of necessity be responsible for the development and prosecution of uniform, comprehensive plans for the improvement of our larger river systems as a whole, working through and requiring of the individual states, conformity in programs to bring about, most economically, these results.

The Central or Federal Government Agency.—To head these regional, state and metropolitan authorities, a central authority is essential. The lack of such federal agency, clothed with adequate administrative and enforcement powers, is responsible primarily for the slow, unbalanced progress that is now being made in stream pollution control in this country. There are various functions that would natuarlly fall under the jurisdiction of this body.

The integration of the general policy of water conservation in the various regions of the country is most necessary. The interstate boards should be directly responsible to the central authority for the conduct and prosecution of their programs of river systems improvement as well as for the general policies which they adopt in effecting such improvements. Standards of water quality in streams, lakes and coastal areas need to be formulated and harmonized, equable protection of sources of drinking water supplies assured and determination made of the areas where corrective measures are most essential. Coordination with other federal and state governmental agencies dealing with various phases of our water resources is also highly desirable. The improvement of navigation, the creation of water power by impounding reservoirs and lakes, the construction of comprehensive flood control projects and the extensive work of soil erosion prevention are only a few of the activities now under way on a national scale that vitally affect any stream pollution preventive measures that may be undertaken. The constituted federal agency should have ample powers to coordinate these various activities to the end that conflicts in aims and accomplishments in the improvement of our public waters be avoided to the largest possible extent.

A central federal authority would seem desirable for the continued stimulation of state and regional authorities to effective action. Such stimulation would doubtless assume various forms, including those of financial and technical assistance, advisory and consultative service and where necessary, compulsory and enforcement action. Doubtless the quickest and most immediately effective stimulus to regional or local authorities would be financial grants of federal funds. The extent, if any, of such assistance and the manner of its application is a subject not within the province of this discussion, but if such aid is made available by the Federal Government, it would seem that a competent, well informed central authority should be charged with its administration and distribution. Technical and legal assistance in organizing and conducting the activities of regional, state and municipal authorities dealing with water pollution problems would likewise stimulate these bodies in many instances. State and municipal officials are frequently assigned such a multiplicity of duties that with the inadequate personnel provided, consistent progress on any specific activity is most difficult, and trained technical staffs can be built up only very slowly. Advisory service too is frequently most helpful to such authorities when special and unusual problems arise. There is need, also, for this federal agency to be clothed with definite and positive compulsory powers in the small but nevertheless existing percentage of instances where all other measures fail to bring about co-operation on the part of divisional groups. Need-

less to say, such powers must always be judiciously employed although their existence is generally the only insurance against complete collapse of any governmental authority.

The federal or central body should also have the responsibility of arbitrating differences in objectives and viewpoints of state and interstate agencies and of disagreements that may arise among them. In the administration of extensive programs affecting the interests of many individuals, municipalities and industries, such differences are to be expected. Fair and equitable settlement of such disputes through arbitration provided by the central authority rather than by extended court procedures, should promote the program.

It is also highly essential to provide for a continuing technical group, preferably in the central authority, where intensive research study may be devoted to the solution of the broad problems in water conservation certain to be encountered by the regional, state and metropolitan authorities. Such studies could be undertaken best by the central body because of the freedom from limitations imposed by state boundaries and because by such an arrangement considerable duplication and repetition might be avoided. The facilities and findings of such a research organization should be made available to regional and state groups for the complete investigation of their technical and administrative problems. Such studies would necessarily cover a wide range of subjects and should be so directed as to render efficient service to its clientele. Among many other problems, it would doubtless be found advisable to investigate, (1) improved methods for the treatment and disposal of sewage and industrial wastes, (2) the efficiency of various water purification processes, (3) the practicability of establishing reasonable standards of purity for public water supplies, sewage treatment plant effluents, streams and lakes, bathing beach and bathing pool waters and waters in which edible shell fish are grown, (4) the natural laws of self purification of streams and the extent to which such natural agencies may be depended upon to effect recovery of polluted waters, and (5) the effect upon the public health of polluted water in all its forms as it comes in contact with the human organism. The federal authority would be the logical agency also for the continued collection and dissemination of factual data of value to the watershed, state and municipal agencies concerned with water pollution control. Such information, if authoritative, up-to-date and presented in suitable form would be most helpful and even essential to efficient administration. It would include such matters as technical and administrative practices and procedures; statistics of population distribution, industrial development and occurrence of water-borne disease; and digests of laws and legal decisions relating to the broad problems of water conservation.

These among others are some of the activities that must necessarily devolve upon a central authoritative body created for the specific purpose, if consistent progress is to be made on a national scale in the protection of our water resources from harmful pollution.

Conclusion

The almost universal degradation of our surface waters by polluting substances contributed by population groups and by industry has brought about a general appreciation that comprehensive corrective measures must be put into effect if our streams, lakes and coastal waters are to be of maximum use. Such measures involve the creation and maintenance of governmental authorities that will be required to function over the greater part of the area of our country. Suggestions for the formation and assignment of functions of such an organization are based upon the principle of employing as far as possible the existing machinery of states and of metropolitan areas now in operation, stimulating them where necessary and coordinating them under larger units in order to fit as a whole into the broader program of watershed and river system control. Directing these regional organizations, which are largely concerned with administrative procedures, a central, federal agency is proposed to advance the entire movement on a country-wide basis.

There are many financial and legal angles that must be taken into consideration in the creation and development of such a nation-wide program and which are not discussed here. Their consideration is most essential, however, to the successful functioning of any coordinated authority such as that herein proposed.

These suggestions should not be considered as a final arbitrary plan of organization for control of water pollution. They are suggestions only, designed to indicate some of the factors that should be considered in the formation and operation of a comprehensive country-wide program of water sanitation.

Industrial Wastes

MALT HOUSE WASTE TREATMENT STUDIES IN WISCONSIN

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Introduction

In the process of malting barley for subsequent alcoholic fermentations, large quantities of wastes of a pollutional character are produced. In cases where this waste is discharged into relatively small streams a condition of gross pollution is apt to occur. Stream surveys have shown that the wastes from malthouses in Wisconsin have seriously polluted streams, indicating the necessity of installation of some type of waste treatment plant. A search of literature failed to reveal any detailed information on the subject of malthouse waste treatment. The waste, containing large amounts of organic matter in solution but very little in suspension, suggested oxidative treatment such as supplied by activated sludge or trickling filter systems. In view of the character of the waste and other considerations to be discussed, the trickling filter treatment was selected for trial on an experimental basis. A cooperative plan was made between the owners of the malthouse and the Bureau of Sanitary Engineering of the Wisconsin State Board of Health for the construction and operation of an experimental trickling filter. Following preliminary studies, the plant was constructed during August and operated continually until the middle of December, 1934.

This experimental work was carried out as a part of the comprehensive program of the State Committee on Water Pollution to secure effective methods for utilization or treatment of industrial wastes. The results indicate that the trickling filter system affords a satisfactory means of treatment for this type of waste. A full scale installation designed on the basis of the information obtained is to be constructed at the malthouse where the work was conducted.

Malting Process

The process of fermentation of various grains and vegetables with the formation of alcohols depends upon the action of enzymes. The

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purpose of malting barely is to produce the enzyme "diastase" in such a quantity that only a small amount of malt is necessary as an inoculant in the manufacture of beer and like products. Diastase is the enzyme of malt which is active in converting starch into soluble starch, dextrin and eventually maltose.

The process of manufacturing malt from barley at the plant where the work was conducted, is briefly as follows: The barley is placed in a large hopper bottomed cyclindrical steel tank where it is kept submerged in cool water for a period of about three days. The function of this portion of the process, which is known as "steeping," is to impart moisture to the grain and to remove undesirable coloring matter and extractives from the husk. During the three day steeping period the water is changed four times, this steep water forms a large portion of the pollutional waste produced. (See flow sheet, Fig. 1).

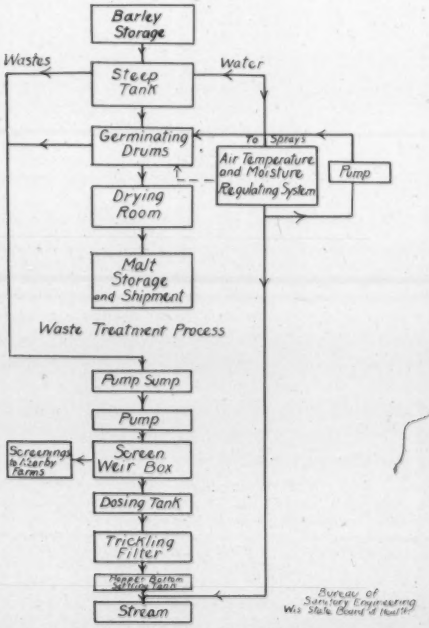


FIG. 1. Malting and waste treatment process flow diagram showing sources of waste. Bureau of Sanitary Engineering, Wisconsin State Board of Health.

After the barley has been properly moistened in the steep tank, it is placed in a large, slowly rotating cylindrical drum, called a germinating drum, where it remains for about five days. During this period of germination the formation of enzymes occurs with the concurrent liberation of heat and carbon dioxide. It has been found that if the carbon dioxide content of the air in the drums exceeds 3 per cent or if the temperature becomes excessively high the rate of germination will be retarded, and to avoid this, cool moist air is continually blown through the drums. After the malt has been in the drums for three days of the five-day period, water is sprayed over it for from one and one-half to three hours, a large portion of this water is later drained off. This germination drum water forms the second largest quantity of pollutional waste.

Upon completion of the germinating process the malt is removed to a series of drying floors where it is slowly dried to a sufficiently low moisture content that it can be stored or shipped.

Source and Quantity of Waste

The waste produced in the malting process consists of water from the steeping and germinating tanks together with cooling water from the air tempering room. Only the first two wastes contain pollutional material and therefore require treatment. In order to determine the quantity of pollutional waste discharged, a weir was placed in the sewer receiving the waste from the steep tanks and germinating drums. A series of six tests each of twenty-four hours duration were made, weir readings being taken every 30 minutes.

The results of these studies are tabulated in Table I, and indicate that, on the average, seventy-five gallons of pollutional waste were produced per bushel of barley processed. The maximum quantity obtained during the six runs was eighty-nine gallons of waste per bushel of barley while the minimum was sixty-seven gallons. The data further indicated that at certain times during the twenty-four hour period, the rate of discharge was as great as 260 per cent of the average, this occurring when the steep tanks were being drained. While the volumes of waste per bushel of barley obtained during these tests were reasonably consistent, it is believed that each plant should be considered as a separate problem.

Character of Waste

At the time the determinations of the quantity of waste discharged by the plant were made, a series of composite samples of the waste were collected at thirty minute intervals in quantities proportional to the rate

TABLE I.—*Quantity and Chemical Characteristics of Malthouse Wastes*

Run No.	Total Gallons of Pollutional Waste per 24 Hours	Bushels of Barley Malted	Gallons of Waste Per Bushel of Barley	5 Day B.O.D. of Waste in p.p.m.	Total Organic Nitrogen in p.p.m.	Nitrogen as Free Ammonia in p.p.m.	Total Solids in p.p.m.	Total Suspended Solids in p.p.m.	Oxygen Consumed in p.p.m.
1	443,000	6165	72	385	16.0	6.9	1069	80	278
2	515,000	7245	71	330	12.0	3.5	872	52	220
3	482,000	7190	67	460	14.0	4.1	1000	110	330
4	602,000	6780	89	370	10.0	5.5	864	62	312
5	527,800	7245	73	330	13.0	9.4	876	56	314
6	565,800	7350	77	455	—	—	—	—	—
Av.			75	390	13.0	5.9	935	72	291

of flow, and analyzed in the State Laboratory of Hygiene. The results obtained from these analyses are given in Table I. The five-day B.O.D. of the plant waste averaged 390 p.p.m., the maximum being 460 and the minimum 330 p.p.m. The total organic nitrogen averaged 13.0 p.p.m., and the oxygen consumed value averaged 291 p.p.m. A study of the solids data indicates that the organic matter in the wastes is chiefly in solution. The total soluble solids averaged 863 p.p.m., of which 328 p.p.m. was mineral matter in the water supply, while the total suspended solids averaged only 72 p.p.m.

In addition to the above studies, a sample of the waste produced by each step in the malting process was collected and analyzed. The results of analyses of these samples are given in Table II. It is apparent from these analyses that the first steep water has the greatest

TABLE II

	5 Day B.O.D.	Total Organic Nitrogen	Free Ammonia Nitrogen	Total Solids	Total Volatile Solids	Total Suspended Solids	Hours in Contact with Malt
First Steep Water	960	69	16.5	4856	1968	176	24
Second Steep Water	920	72	16.2	2372	1382	104	12
Third Steep Water	185	4	3.5	418	194	32	12
Fourth Steep Water	254	7	3.1	452	244	24	16
Germinating Drum Water	50	12	7.2	534	274	80	—
Germinating Drum Wash Water	—	9	2.4	456	222	136	—
Water Supply	—	—	—	328	125	—	—

pollutional strength, the five-day B.O.D. being 960 p.p.m., the total solids 4,856 p.p.m., the total organic nitrogen 69 p.p.m., and the free ammonia nitrogen 16.5 p.p.m. The pollutional strength of the steep waters decreases as the steeping process continues, the second steep water having a B.O.D. of 920 p.p.m., the third 185 p.p.m., and the fourth 254 p.p.m. The fourth steep water has a somewhat higher B.O.D. than the third; this is apparently due to the fact that the period of contact with the barley is longer in the fourth steep. The germinating drum water and the drum wash water have a low pollutional strength.

Experimental Treatment Plant

Prior to the construction of the waste treamtent plant a survey was made to determine not only the nature and volume of wastes produced but also possible treatment processes. The survey indicated sedimentation alone would be ineffective because the major portion of the pollutional material is in solution. Small-scale laboratory experiments indicated that biological oxidation offered a promising method of treatment. The trickling filter, however, was selected as offering a practical method for the solution of this particular problem. Experience has shown that trickling filters will produce good results in the treatment of somewhat similar industrial wastes with relatively little attention necessary for proper operation and maintenance.

The rock filter used had a depth of eight feet, and an effective diameter of nine feet. Crushed limestone of a size that would pass a one-inch screen and be retained on a two-inch screen was used as the filter media. The waste was delivered by means of a centrifugal pump to and elevated weir stilling basin and filter dosing tank. (See Figs. 1 and 2.) The waste was applied intermittently to the filter by means of a dosing siphon and rotary distributor. (See Fig. 3.) A small hopper-bottom secondary settling tank further treated the filter effluent. The volume of waste applied was measured with a triangular weir, the quantity applied being regulated by means of an overflow pipe in the weir box.

Method of Conducting Tests

The method of conducting these studies consisted of operating the filter at a fixed dosage rate for a period of one week, composite samples being collected only on the sixth and seventh days of this period. By following the above procedure the biological life on the filter had a period of five days to become adjusted to the given dosage rate. It is felt that by following this procedure rather than collecting samples shortly after the dosage rate was changed, the results obtained more nearly approach those that could be expected at the rates used with continued operation.

FIG. 2.—Experimental plant for the treatment of wastes from a malthouse in Wisconsin: (*a*) Raw waste pump sump; (*b*) pump house and laboratory; (*c*) screen, weir box and dosing tank; (*d*) trickling filter; (*e*) secondary settling tank and effluent flume.

FIG. 3.—Rotary distributor used for applying the waste uniformly over the surface of the trickling filter.

Composite samples were collected for a period of six hours at 30-minute intervals from the filter influent and effluent, and from the secondary settling tank effluent. Samples were also taken from collecting pipes at two, four and six-foot depths in the filter.

Discussion of Data

Suspended matter in the wastes, consisting mostly of grain and sprouts produced in the germinating process, was removed by a 20-mesh screen prior to discharge to the trickling filter. The removal of such suspended matter was considered essential to prevent clogging of the distributor and filter bed. It was noted that the sprouts tended to clog the screens, necessitating frequent cleaning. From this and other observations, it is believed that a rotary screen provided with needle sprays, and possibly mechanical cleaning facilities, will need to be provided as a part of any trickling filter system for the treatment of malt house wastes. Screenings may be utilized as stock feed on nearby farms.

Tests were continued into December to ascertain the effect of low temperatures upon treatment efficiencies and to determine whether serious operating difficulties would be encountered with an uncovered filter for malt house wastes. Observations made indicated that irrespective of the air temperature, from 16° to —7° C., the raw waste remained at about 14° C. The drop in temperature of the wastes in passing through the filter was slight, being only four degrees when the air temperature was —7° C. Filter efficiencies did not appear to be materially decreased and no difficulty was experienced with ice accumulating on the surface of the filter under winter operating conditions experienced during these studies.

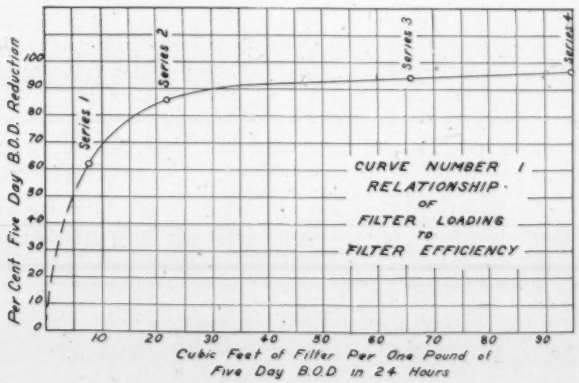
A study of analyses obtained indicated that the most satisfactory design unit would probably be the B.O.D. loading, calculated as cubic feet of filter per one pound of five-day B.O.D. per 24 hours.

The five-day B.O.D. data were grouped into four series, the runs in each series having approximately the same loading. The number of individual tests of each series are given in Column 2 of Table III. The data of Table III are shown graphically in Curve 1, which indicates that the percent B.O.D. removal by the filter increases rather rapidly as the pollutional loading decreases until a "critical loading" is reached, after which an additional decrease in loading produces a relatively small increase in filter efficiency. The curve indicates that a reduction of about 90 per cent in the five-day B.O.D. of the waste can be expected for a filter loading of one pound of five-day B.O.D. per 30 cubic feet of filter per 24 hours. A curve having characteristics similar to Curve 1 was found to apply for trickling filters treating milk plant wastes. It is interesting to note, however, that in the treatment of milk wastes

TABLE III

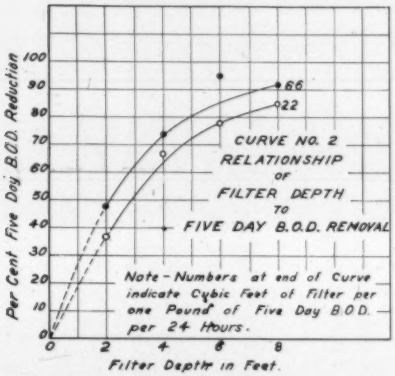
Series No.	Number of Runs Averaged	Filter Influent	Filter Effluent	% Reduction	Final Effluent	% Reduction	Cubic Feet of Filter per 1# of 5 Day B.O.D. per 24 Hours
1*	7	490	210	57	190	61	8.6 *
2	6	250	38	85	35	86	22
3	3	340	24	93	19	94	66
4	1	300	12	96	8	97	95

* Five Day B.O.D. Data



a five-day B.O.D. reduction of 85 per cent requires about 80 cubic feet of filter volume per 1 pound of B.O.D. per 24 hours.[1] A series of studies conducted in the laboratory using the activated sludge process also indicated that the B.O.D. could be rather easily reduced.

In addition to the studies of filter loading, reductions in the pollutional strength of the waste at different depths as it passed through the filter were determined, results being shown in Curve 2. Data indicated that the rate of removal of B.O.D. by the filter was not uniform, but decreased with increased depth; that is, the percentage removal was greatest in the upper portions of the filter. The curves for loading of 66 and 22 cubic feet of filter per one pound of B.O.D. per 24 hours



indicate that the bottom two feet of the filter reduced the pollutional strength of the waste only about five per cent.

These experiments were conducted with a relatively small filter with open sides, thus affording excellent ventilation at all times (see Fig. 4), and the filter was receiving more attention than might be expected with

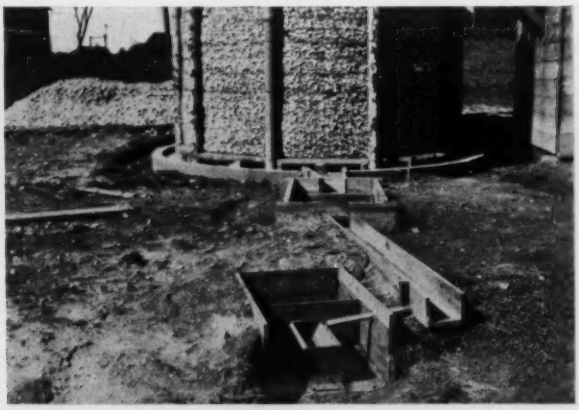


FIG. 4.—Construction details of experimental filter for malthouse wastes, showing secondary settling tank and tip trough sampler in the foreground.

a full-scale plant. For the above reasons it is felt that these results may be somewhat better than could reasonably be obtained with a filter to take care of all the wastes. In view of the above, together with the desirability of a factor of safety to take care of the unexpected loads which may occur at some time in the future, the full scale plant is designed on the basis of 50 cubic feet of filter per one pound of five-day B.O.D. per 24 hours.

The pH of the influent to the filter averaged 6.7 while samples from the 6 and 8-foot levels averaged 7.7 and 7.9, respectively.

Biological Observations

The malthouse wastes appeared to have a characteristic common to the wastes produced in fermentation processes generally in that they are conducive to growth of fungi.[2] The surface of the filter carried an abundant growth of the fungus, *Fusarium*, at all times, while *Pythium*, another fungus growth, appeared in the sewer leading from the malthouse and in the pipe lines to the filter. This latter growth occasioned difficulty in that it tended to clog the screen used to remove the sprouts and barley from the waste.

After the filter had been operated at excessively high dosage rates for a considerable period of time a rather abundant fungi growth was observed in the effluent channel, further indicating that at high dosage rates the degree of purification desired was not being accomplished.

A microscopic study of the filter slime showed it to be abundant with protozoa and other lower forms of life common to trickling filters.

During a portion of the tests the filter was sloughing. The slough-off settled rapidly and was brown and granular in appearance, looking very much like the slough-off from domestic sewage filters.

Conclusions

From the data obtained during the studies the following conclusions can be drawn:

1. A properly designed, constructed and operated screening and trickling filter system can be successfully used to treat malthouse wastes.

2. The combined pollutional wastes produced at the plant where studies were conducted had an average five-day B.O.D. of approximately 400 p.p.m., a large portion of the pollutional material present being in solution.

3. About 80 gallons of pollutional waste are discharged per bushel of barley processed.

4. A filter designed on the basis of fifty cubic feet of filter per pound of five-day B.O.D. per twenty-four hours and a depth of from six to eight feet can reasonably be expected to produce a 90 per cent reduction in five-day B.O.D.

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Editorial

SEWAGE ANALYSIS

The report of the Committee of the Federation on Standard Methods of Sewage Analysis is probably the most valuable of the various Federation reports, most of which have been suggested and fostered by our Chairman, Mr. Charles A. Emerson, Jr. The value of the Standard Methods report will be appreciated by many chemists as a guide to standard laboratory practice and particularly as a stimulus toward research on analytical methods, resulting in improvement and expansion of current methods of analysis.

Our Committee report is not intended to supplant the Standard Methods of the American Public Health Association, which Association, as stated by Dr. Hatfield (page 445), will probably accept the Federation report, with minor modifications if found desirable, as the section on sewage analysis in the next edition of the A. P. H. A. Standard Methods. However, there has appeared to be a need for more intensive work on sewage analysis than has in the past been provided by the A. P. H. A. This need was outlined in a written symposium organized by the Editor of THIS JOURNAL and printed in the July, 1930, issue. In this symposium, some of the deficiencies of the then Standard Methods relating to sewage analysis were pointed out, and as a result of the assembled statements of various well-known chemists, experienced in sewage analysis, a number of useful procedures were outlined to serve as a basis for improved and more comprehensive methods. Some of these procedures were: a method for estimating volatile or putrescible solids in grit; various methods for determining settleable solids; and the selection of a synthetic dilution water for the B. O. D. determination.

Possibly as a result of this symposium, but more likely because of the dynamic influence of Harry E. Jordan, Chairman of the Joint Editorial Committee of the A. P. H. A. and the American Water Works Association, the 1933 Edition of Standard Methods contained an enlarged section on sewage analysis and also several provisional methods, which had not appeared in previous editions. In fact, prior to the Fifth Edition, in 1923, there had been no separate section on sewage analysis, and the first separate section was prepared by the writer for that edition. The section has grown, but not so rapidly as desirable, in view of the great developments in sewage treatment during the past twelve years.

The Federation of Sewage Works Associations is the logical agency that should be responsible for development of and research on methods

of sewage analysis. Therefore the report of Dr. Hatfield's Committee is the foundation for an important function of the Sewage Works Federation—a continuous and expanding program of research and improvement of methods of sewage analysis. It is doubtful whether such a program can emulate the highly organized systems of the Association of Official Agricultural Chemists or of the American Society for Testing Materials, but their systems might well be followed, on a simpler basis, for the future work of our Standard Methods Committee.

Only those who have collaborated in such undertakings can realize the magnitude of such a program, but all members of the Federation can appreciate the work that Dr. Hatfield has put into the present report. Many of the members of his committee have also given considerable time to the work. The Chairman, however, usually does the lion's share. (The Federation also owes a debt of gratitude to Prof. Earle B. Phelps for the time he spends on the preparation of the annual reports of the Research Committee.)

As noted on page 491, the Report of the Standard Methods Committee is to be reprinted and copies may be purchased from Mr. Buffum's office. Separate copies may be desired for laboratory use, as being more convenient than use of this rather voluminous issue of the Journal. It is hoped that the methods will be used, studied, improved and augmented by all chemists engaged in sewage analysis, and further that comments on procedures recommended by the Committee be sent as soon as possible to Dr. Hatfield so that he may make any necessary corrections or additions in his report to the Committee on Standard Methods of the A. P. H. A.

In the interest of standardization, it would be unfortunate if the various local associations of the Federation should attempt to formulate standard methods for their own use, and likewise the publication of various laboratory manuals by individuals might lead to confusion and unnecessary duplication. For these reasons, it is hoped that all associations affiliated with the Federation will lend their support to the future work of the Federation Committee on Standard Methods of Sewage Analysis, thus making it the authority on such methods, subject to the stamp of approval of the present universally recognized authority, Standard Methods of Water Analysis of the American Public Health Association.

F. W. M.

Proceedings of Local Associations

MICHIGAN SEWAGE WORKS ASSOCIATION

ELEVENTH ANNUAL CONFERENCE

East Lansing, March 25–29, 1935

The Eleventh Annual Meeting of the Michigan Sewage Works Operators Association was held at Michigan State College, East Lansing, Michigan, March 25–29, 1935. The first three days were devoted to instruction in laboratory procedure, both in chemical and bacteriological analyses. This was carried on in the form of lectures and demonstrations followed by actual work in the laboratory. Lectures on plant hydraulics were also given. This work was carried on by Professors Eldridge, Mallmann, and Theroux of the Michigan State College staff. Assistance in laboratory instruction was given by Mr. Welch, Mr. Mudgett and Mr. Damoose, who conduct the analytical work at the Flint, Pontiac and Grand Rapids sewage treatment works. The work was very enthusiastically carried on as evidenced by the hours of study put in. Although some of the evening sessions did not close until 11:00, there was always a group waiting to begin work the next day before 8:00 in the morning. The short course enrollment was 34, and the full attendance at school and conference was 82.

Dean H. B. Dirks gave a very cordial invitation to the Association to make full use of the college facilities and assured the members of the sincere interest of the college in their welfare.

Edmund C. Miller, Engineer in Charge of Sewage Treatment at Dearborn gave an interesting talk on the recent additions to the West Side Dearborn Plant. This is the "Laughlin Process" plant which has received considerable publicity during the past two years. More efficient chemical mixing and flocculating apparatus has been installed, and the inlet piping to the clarifiers changed. An incinerator for burning sludge has been constructed and has been in operation for about 60 days. Mr. Miller feels that the installation will be entirely satisfactory although the period of operation is too short to give more than an indication of possible economy and operating results.

Mr. R. A. Anderson of Muskegon Heights presented a paper on the operating experiences of the plant at that city. This is an activated sludge plant using combined air blowing and paddle wheels for aeration. This paper was particularly interesting as it presented the ex-

periences in operation of a plant, rather than the design or planning viewpoint.

Maintenance of electrical equipment was discusesed by Mr. W. H. Collins of the Lansing Board of Water and Light Commissioners. Mr. Collins' talk brought out the desirability of careful selection of the proper type of electrical equipment for specific services, the importance and methods of maintenance, and a system of records for systematically controlling the maintenance of such equipment. Mr. Collins' talk was intensely practical in nature and of great value to all plant operators.

Prof. W. C. Hoad of the College of Engineering, University of Michigan, spoke on the topic "Tank Treatment of Sewage." Professor Hoad outlined the various types and characteristics of sedimentation tanks, bringing out the advantages and shortcomings of each type. He presented a series of charts showing the actual results obtained at several plants.

With the greatly increasing numbers of plants using chlorination Mr. R. J. Faust's discussion of "Maintenance of Chlorine Equipment" was very timely. Mr. Faust is Assistant Engineer with the Michigan Department of Health. Several slides showed very clearly the construction of the various kinds of chlorinators and made it easy to understand how they work and the reasons for certain difficulties in their operation.

The City Manager of Royal Oak, Mr. E. M. Shafter, discussed the relations between the plant operator and the city administrator. He particularly brought out the fact that the City Manager must rely on the plant operator for information as to plant performance, maintenance requirements, necessity for extensions and replacements and budget estimates, and all such information should be accurate and completely reliable.

A paper on some of the interesting phases of operation of plants in Michigan was given by W. F. Shephard, Assistant Engineer, Michigan Department of Health. This paper was primarily based upon the results of a survey of some 20 plants made in 1934 by the Department of Health.

A paper on the operation of sewer rental law in Ohio was to have been presented by Mr. Floyd Brown of Marion, Ohio. Mr. Brown was unable to attend personally but very kindly forwarded his paper.

Prof. W. L. Mallmann presented an interesting discussion of chlorination of sewage with small amounts of chlorine, in relation to bacteriological content. Results of experimental runs at the Traverse City plant with varying percentage of chlorine demand satisfaction and contact periods were presented.

The conference group inspected the new water softening plant serving the City of East Lansing.

Dr. C. C. Slemons, State Commissioner of Health, was the speaker at the annual banquet.

At the annual business meeting the following officers were elected: Mr. N. G. Damoose, Grand Rapids, President; Mr. R. A. Anderson, Muskegon Heights, Vice-President; Prof. W. L. Mallman, East Lansing, Director 1935–36; W. F. Shephard, Michigan Dept. of Health, Lansing, Sec.-Treasurer; Mr. E. C. Miller of Dearborn holds over as Director for the ensuing year.

ANNOUNCEMENT

The SEWAGE WORKS JOURNAL takes pleasure in announcing the affiliation with the Federation of Sewage Works Associations of the Kansas Water and Sewage Works Association with a charter membership of 23, and that of the Northwest Conference on Stream Pollution, with a membership of 18. The first meeting of the latter group was held in May at Lewiston, Idaho.

NOTICE

The Ninth Annual Conference of the Pennsylvania Sewage Works Association will be held at State College, Pennsylvania, June 24, 25 and 26. A varied and interesting program, including an operators' symposium, has been planned.

Reviews and Abstracts

SEWAGE EFFLUENTS

By A. Sciver

The Surveyor, 87, 410 (March 22, 1935)

The official standards for sewage effluents employed by the Thames Conservancy and the Port of London Authority as secured by the author, are given in the current issue of the Journal of the Institution of Sanitary Engineers. These, as well as those promulgated by the English Royal Commission on Sewage Disposal, are:

Limiting Standards for Sewage Effluents
(Constituents in parts per million)

Authority	Albuminoid Ammonia [2]	Suspended Matter	D.O. Absorbed in 5 Days, 65° F.[2]	O. Absorbed from Acid Permanganate in 3 Hrs., 98° F.[2]
1. Eighth Royal Commission on Sewage Disposal	—	30	20	—
2. Thames Conservancy [1]	2	30	20	—
3. Port of London Authority				
(*a*) Above London Bridge	—	30	20	—
(*b*) London Bridge to Long Reach				
(1) Except non-tidal tribs.	5	—	—	50
(2) Non-tidal tribs.	—	30	20	—
(*c*) Long Reach to Lower Hope Beacon	7	—	—	70
(*d*) Lower Hope Beacon to Seaward	Absence of offensive conditions.			

[1] Standards are not entirely rigid but may be modified if justified by special circumstances. An albuminoid ammonia figure of 30 p.p.m. is accepted below the water intakes at Hampton and above Teddington Lock.

[2] Including suspended matter in the sample.

The Port of London Authoirty results can be expressed as an "impurity figure" calculated as follows:

$$\frac{(\text{Albuminoid Ammonia} \times 100) + (\text{Oxygen absorbed} \times 10)}{2}$$

Thus in the river section from London Bridge to Long Reach the impurity figure must not exceed 50 to be considered as satisfactory, while in the stretch from Long Reach to Lower Hope Beacon, this limit is increased to 70. These standards are subject to review at any time by the Port of London Authority.

J. K. HOSKINS

ELIMINATION OF SMELLS FROM SEWERS AND SEWAGE DISPOSAL WORKS

By HAROLD WILSON

Paper presented at the Bristol Congress of the Royal Sanitary Institute, reproduced in the January, 1935, issue of the Institute's Journal and reviewed in *The Surveyor*, **87**, 223–24 (February 8, 1935).

The odors arising from sewage disposal works and from sewers are accepted resentfully by the public and, while they do not carry pathogenic organisms, they do have a constant depressive effect on the nervous system and so must be regarded as detrimental to health. The author, using the Johannesburg, South Africa progress as an example, shows that it is possible to eliminate such smells at reasonable cost.

Odors are intensified and carried greater distances by moisture-laden air currents such as those passing over sewage treatment works or through sewers. Examples of this effect may be observed in the case of odorless dried sludge that is moistened, or of polluted streams that can be detected miles away on summer evenings.

Odor Prevention in Sewers.—Sulphide odors in sewers can be prevented most successfully by maintaining alkaline conditions throughout the sewage and disposal system. The degree of alkalinity is important, as too high alkalinity is detrimental. Lime alone or in combination with chlorine gas, added at carefully selected points in the sewerage system, is cheap and effective. This method of adding coagulants has the further advantage of providing gentle agitation essential to the proper flocculation of chemicals with colloidal impurities while the sewage is approaching the treatment plant. At one of the Johannesburg plants milk of lime is discharged to the sewer 1¼ miles above the outfall works, and produces a rapidly settling floc in the sewage by the time it reaches the treatment works.

Hydrogen sulphide is almost instantaneously destroyed by chlorine. The addition of ozonized air or ozonizing the sewer air itself are also effective for eliminating this nuisance.

Odor Control During Treatment.—The screening plant should be enclosed and screenings incinerated or digested, following mechanical

disintegration, without exposure to the outer air. Proper grit washers should eliminate smell from grit collectors. During sedimentation, there should be the least exposure of surface possible to the air and prompt removal of sludge and surface scum. For these reasons, shallow, rectangular, horizontal flow tanks are unsuitable, even when equipped with travelling or revolving sludge and scum removers. The author, therefore, favors the hopper type or Dortmund tanks. These deep tanks present less surface, can be easily covered, facilitate complete sludge removal, and reduce scum formation to a minimum, although they do have the disadvantage of extra cost and production of a more watery sludge.

The screens, detritus tanks, sedimentation tanks and sludge well if inclosed in a sealed building, will confine odors from these devices. Smells from other devices such as sewers, screen housings, sedimentation tanks, etc., may be eliminated in several ways. Such foul air may be passed through the aeration tank of the activated sludge plant because activated sludge absorbs almost immediately the odors from sewage with which it is mixed. This may be effected by locating the compressor intake inside the building over the sludge well as was done at Johannesburg.

Another method that proved satisfactory was to pass this foul air over activated carbon scrubbers. A scrubber containing ¼ cubic yard of activated carbon completely deodorized 10 million cubic feet of air but revivifying the carbon presented a difficulty. It was later found that pressure ozonization of the foul air was effective and cheaper. A large building has now been erected over the Dorr clarifiers and sludge well and has been connected to the screen house. The foul air is passed through Ozonair apparatus installed in the screen house and is then discharged, deodorized, into the open air.

The Delta sewage works at Johannesburg now under construction are designed on the above lines to eliminate odor nuisance. The entire plant is estimated to cost £100,000, £5,000 of which, or 5 per cent, is to be charged to tank and other equipment housing including Ozonair apparatus.

J. K. Hoskins

SOME IMPORTANT CONSIDERATIONS AFFECTING THE DESIGN OF SEWAGE TREATMENT WORKS

By Granville Berry

The Surveyor, **87**, 399–400 (March 22, 1935)

New demands upon proper design of sewage disposal works is resulting from the tendency to provide treatment for sewage flows of entire

drainage areas at some single convenient point and thereby eliminate many smaller and frequently out of date plants. A difficulty in providing such design is the lack of information as to tank capacities, etc., for which allowance should be made. The author brings together observations on the best engineering practice and experience applicable to British conditions, in this paper.

Dry weather, per capita flows, industrial waste contributions and subsoil water infiltration into the sewer system are factors that profoundly affect the treatment plant design. The total volume of flow to be treated fully is also of importance. The old conception, satisfactory to the Ministry of Health, of 3 or 2 times the dry weather flow plus 1 1/10 trade waste for the combined system and the separate system respectively would seem to be in need of revision upward in view of increased volumes of both trades waste and infiltration in local instances.

Selection of the treatment process should be based upon consideration of the degree of purification to be effected, and characteristics of the sewage to be treated, and not only upon a recommendation that the process has been employed with success elsewhere.

Detritus Tanks.—Deep pyramidal detritus tanks are being supplanted by long, shallow, flat bottomed grit channels, of a size governed by the expected frequency of cleaning and estimated quantity of grit retained. Failure to maintain the optimum velocity of flow of 1–1½ ft. per second results in the detention of objectionable organic matter with the grit, subsequently giving rise to aerial nuisance.

Screens and Screenings.—Older forms of screens were frequently too fine and retained too much organic solids. Recently coarser screens are being employed which pass such material to the tanks. Although, for avoidance of offensive nuisance, screenings are more carefully disposed of, the potential fertilizer value of screenings is not generally realized. Indications are that soon screenings will be disposed of by use as fertilizer base or by mixing with the digesting sludge in the sludge digesting tanks.

Settling Tanks.—Deep, upward flow Dortmund tanks and devices for rapid and efficient removal of sludge are notable developments which permit reduction in tank capacity with resulting shorter detention periods. There is need for some definite guide for use as a basis of design of this newer type of tank. The nature and strength of sewage to be dealt with and the settleable solids content to be removed would be factors in such a guide.

Biological Processes.—"The percolating filter remains today as one of the most valuable and reliable stages on most sewage purification works." Its primary function is biological rather than one of filtration and the process is unchanged although the advent of automatic syphons,

central draw-off, solid wall filters, film sprays, etc., are effecting still higher efficiencies in purification.

Further information is needed on the types of sewage that respond readily to treatment by the activated sludge process. The possibilities of satisfactory and economical performance as a stage process in conjunction with preliminary or subsequent treatment need to be explored.

Humus Tanks.—These tanks are now designed for four hours dry weather flow, double the previous capacity provided.

Sludge Digestion.—The time has come for the adoption of standard practice, with limitations, in order that local authorities may provide proper design of tanks, types of processes that will insure reliable results and the degree to which digestion should be carried. Temperature, pH control and stirring can be dealt with as local conditions may require if these items are properly provided for in the design.

J. K. Hoskins

THE AGRICULTURAL USE OF SEWAGE SLUDGE

By Techn. Oberinspektor Gouder

(Superintendent, Sewage Treatment Works, Frankfort-on-the-Main)

Gesundh.-Ing., 57, 461 (1934)

Recently an effort has been made to increase the yield of farm products and as a result more fertilizer is needed. Sewage sludge is valuable as fertilizer for its nitrogen and phosphorus as well as for its humus. About 0.18 gallon of sludge, containing about seven per cent dry solids, is obtained per capita per day. The nitrogen content amounts to about two per cent of the dry solids and the soluble phosphate content is almost as much. The cost of 220 pounds of nitrogen is 85 marks and of phosphoric acid (P_2O_5), 35 marks. The annual value of these fertilizing materials from 100,000 inhabitants amounts to about 30,000 marks for nitrogen and 12,000 marks for phosphoric acid.

The main objections to the use of sewage sludge as fertilizer are: the excessive water content, odors, the transfer of odors to the crops, the increased amount of weeds, the danger of infection and the high cost of collecting the sludge. Undoubtedly, the most serious disadvantage is the large water content, which prohibits transportation for long distances. The reduction of water content by treatment in beds, centrifuging, pressing and concentrating by heat or chemical precipitants failed because of the high cost, lack of space and the difficulties in removing the solids from the water. For some years a new type of centrifuge has been satisfactorily used at the Frankfort treatment

plant. The machine is automatic and handles about 2640 gallons of sludge per hour, reducing the moisture content from about 92 to 75 per cent. The centrifuged sludge is piled six feet deep, alternately on two beds, each 37,500 square feet in area. After three to four months storage the sludge is dry enough to spade. To avoid nuisance from odors, small beds are recommended and the sludge should be covered with one to two inches of peat. This dried sludge is sold for fertilizer purposes at a price of 0.30 mark for 35 cubic feet. The centrifuge liquor contains from 1.0 to 1.5 per cent of very finely divided solids. The assertion that sludge affects the odor of the farm produce was refuted by the experience of farmers near Frankfort. To prevent the increase in weeds, the cultivation of vegetables requiring hoeing, such as sugar beets and potatoes, is recommended for the first two years and cereals for the third year. By hoeing, the weeds are soon exterminated, since they are annuals and not perennials. Experience shows that there is little danger of infection from sewage sludge.

G. P. Edwards

A NEW ACTIVATED SLUDGE PROCESS FOR PARTIAL TREATMENT

By Dr. Leiner

Gesundh.-Ing., **57**, 323 (1934)

The activated sludge process produces the highest biological purification if a sufficiently long aeration period (four to eight hours) is used. Attempts to shorten the aeration period have failed because the activated sludge quickly loses its purifying power.

It is known that aeration of sewage and only a small amount of healthy activated sludge for one-half to one hour produces satisfactory purification. In aeration tanks the greatest purification occurs during the first hour. From the first to the second hour it is essentially less and from the second to the eighth very little purification takes place. About five per cent of activated sludge will give good purification, although 12 to 30 per cent gives the best results.

In the new process the aeration tanks are divided into two groups: the conditioning group and the overloading group. In the conditioning group only enough sewage is added to keep the activated sludge healthy and capable of purification and the remainder of the sewage flows to the overloading tank. Sludge is returned from the conditioning tank to both the conditioning and overloading tanks and the sludge from the overloading tank is wasted. The longer the aeration period the greater

is the saving in space and power. With an aeration period of four hours, saving in space amounts to three-eights of the total volume and with eight hours aeration the saving is seven-sixteenths. The cost of construction, interest, depreciation, maintenance and power are reduced to about one-half of similar costs for complete purification, without making the operation more difficult to supervise and without essentially reducing the degree of purification. Experience has shown that if the total amount of sewage is divided into two equal parts for conditioning and overloading, the purification in the overloading tank amounts to about 80 per cent of that in the conditioning tank and the purification for the entire amount of sewage is only ten per cent less than for complete purification. Based on the B.O.D. reductions, purification by trickling filters is about ten per cent poorer than by activated sludge. Using the partial activated sludge treatment for one-half of the sewage, the average total purification is at least as good as with trickling filters and is usually sufficient for most cases.

A great advantage of this process is the flexibility of operation. Any undesirable addition of sewage is sent to the overloading tanks. Almost immediately the efficiency decreases, the sludge becomes gray and the entire overloading process seems to be destroyed but as soon as the additional loading is discontinued the color and efficiency return with the same rapidity.

The construction cost of trickling filters is very high but the power requirements are lower than for activated sludge, especially that operated by compressed air. The new partial activated sludge treatment reduces the power requirements so greatly that trickling filters deserve attention only in small towns having highly concentrated industrial wastes or where the filters can be operated by gravity without pumping.

G. P. EDWARDS

THE WASTES FROM CELLULOSE AND PAPER INDUSTRIES

BY DIPL.-ING. STEINSCHNEIDER

City Director, Aschaffenburg, Bavaria

Gesundh.-Ing., 57, 638 (1934)

The paper industry can be divided into the manufacture of the following kinds of pulp:

1. Sulphite cellulose
2. Soda and straw
3. White

4. Steamed mechanical wood
5. Rag
6. Special paper

In the production of sulphite cellulose, wood is heated under pressure in an aqueous solution of calcium bisulphite, which contains an excess of sulphurous acid. The lignins and the sugars in the wood are made soluble and the fibers which remain contain very little fine material. With screens of sufficient fineness and a plant of adequate size the fiber content in the waste can be held down to 12 to 15 p.p.m. In general, sedimentation is not necessary as the added degree of purification is not justified by the cost. About 42 gallons of water are used in the manufacture of one pound of cellulose and an average plant produces 220,000 pounds of cellulose per day. Although the fiber is not important as a cause of stream pollution, some damage may be caused by the dissolved organic material in the waste liquor and the wash water, which may amount to about 2.6 pounds per second from a plant with a capacity of 220,000 pounds per day. This organic material encourages the growth of the lower plants, such as *Sphaerotilus natans* and *Beggiatoa alba* which may cover the banks of a river with a thick coating. The oxidation of these organic substances can cause a serious deficiency of oxygen in the stream. Efforts to eliminate nuisance by treatment in digestion tanks, trickling filters, aeration tanks and by broad irrigation have been unsuccessful.

Since about 50 per cent of the raw material is discharged in the waste, a large amount of work has been done to recover valuable by-products. Part of the sugars can be fermented to alcohol and in Germany today about 13,250,000 gallons of alcohol are produced. The waste can be evaporated and used for fuel but, since about 1100 calories are required to produce one pound of solids, which has a heat value of 1800 calories, the process is not economical. The waste contains more than eight per cent solids.

In the manufacture of soda and straw pulp the fibers are small and clarification tanks are necessary for their removal. The liquid is evaporated to dryness, the organic matter burned as fuel and the sodium salts recovered.

In the production of white pulp the wood is pressed against a rapidly rotating stone and the fine material which results must be removed in clarification tanks. Since this type of plant is small and little waste is discharged (12 gallons per pound of material) stream pollution from this source is not important.

In the manufacture of steamed mechanical wood pulp, the wood is steamed before grinding. As a result, from seven to eight per cent of the wood is dissolved and formic and acetic acids which are produced

must be neutralized with lime before discharge into a stream. The fiber must be removed as completely as possible by clarification tanks.

Since the production of half-stuff from rag takes place only in connection with other processes, the wastes may be combined with other paper wastes for treatment.

In the manufacture of paper the long fibers are separated and the waste contains relatively large amounts of fine material which may be removed continually in a hopper-bottom tank and returned to the paper-making process. In this way the final waste contains only 25–30 mg. of fiber.

G. P. EDWARDS

MEETING OF THE SWISS ASSOCIATION FOR SANITATION AT BADEN-AARGAU

Gesundh.-Ing., **57**, 399 (1934)

(Abstracted from *Schweiz. Bauztg.*, **104**, 9 (1934)

A meeting of the sewage section was held in connection with the twenty-fifth meeting of Die Schweizerische Vereinigung Fur Gesundheitstechnik in Baden-Aargau June 16–17, 1934, at which the following papers were presented:

The Urgency of Stream Protection—Prof. Dr. Steinmann.

The Fight Against Stream Pollution from the Standpoint of the Public Official—Ing. Pelet.

The Relationship Between Sewage Purification and Hygiene—Prof. Dr. Silberschmidt.

View of the Sewage Problems in Switzerland—Dr. Acklin.

The Viewpoint of Fishing Inspectors in Regard to the Practical Questions of Stream Purification—Vouga.

Legislation for the Protection of Streams and the Removal of Sewage—Dr. Surbeck.

Construction of Sewage Treatment Plants—Ing. Steiner. The cost of construction of a mechanical treatment plant amounts to 10–20 francs per capita of the included population. The plant for Zurich in Werdhölzli cost between 17–18 francs per capita.

G. P. EDWARDS

SEWAGE IRRIGATION IN TEXAS

By Earl H. Goodwin

Public Works, 66, 23 (March, 1935)

Irrigation as a means of secondary treatment of sewage has been used in Texas for several years. San Antonio, since 1900, has irrigated 3500 acres. At present, of the 450 sewage treatment plants in the state, about 47 are using irrigation as a means of treatment, approximately 4500 acres being irrigated. The farms vary from 1 to 350 acres, averaging about 20 acres and were purchased at a value of $25–$125 per acre. Cities report a revenue from irrigated crops ranging from $25 to $125 per acre per year.

Two methods of irrigation are in use, subsurface and surface. Although the subsurface method has some important advantages, it is not felt that they justify the extra cost. Three methods of surface irrigation are used: (1) sprays, (2) the border method where the field can easily be divided into strips, (3) the furrow method for row crops. Crops grown are grains, grasses, cotton, alfalfa, nuts and citrus fruits.

Porous sandy soils seem most acceptable for irrigation. Settleable solids should be reduced at least 50 per cent and preferably 80 per cent before the sewage is applied to the land. Sewage containing acids and alkalies is not used. Care must be taken to prevent oil and grease from passing onto the land. Heavy flooding before planting and careful watering during the growing period seem to give the best results. The land is usually plowed to a depth of 12 inches at least once a year. It is laid off in furrows about 3 feet apart and the liquid applied in each furrow in an amount that will be absorbed within 30 minutes. Storage of sewage in earthen reservoirs is necessary at certain periods. These tanks should be placed a sufficient distance from habitation to prevent nuisance and should be carefully maintained. At least 25 per cent of the land under cultivation should be held in reserve for sewage application at such times as the crops would suffer from additional moisture.

Unless efficient supervision is maintained, the sewage and soil may not be properly handled and the crops may not be well selected and properly grown.

R. S. Smith

TREATMENT OF COMMERCIAL WASTES IN CONNECTICUT

ANON.

Public Works, **66**, 49 (April, 1935)

Two years of systematic study of wastes from ten industries have resulted in the following recommended treatments: Neutralization of acid wastes from wool felt manufacture by contact with a high magnesia limestone followed by contact with a high calcium limestone. Heating of candy factory wastes to coagulate the vegetable matter, then skimming, followed by trickling filter treatment. Mechanical screening of cardboard factory wastes and recirculation of the effluent. Neutralization with lime of wastes from finishing small animal skins, followed by sedimentation and filtration. Chemical coagulation of woolen goods manufacture wastes. Screening and trickling filtration or evaporation of distillery wastes and trickling filtration of brewery wastes. Trickling filtration of dairy wastes. Acidification and aeration of cyanide wastes from electroplating.

Promising results have been obtained by catalytic oxidation of some wastes using an inorganic catalyst precipitated on granular substances. The re-use of precipitated coagulant has given good results with certain industrial wastes.

R. W. KEHR

CHEMICAL TREATMENT OF SEWAGE AT DALLAS

BY E. W. STEEL AND P. J. A. ZELLER

Public Works, **66**, 23 (April, 1935)

A study was made of chemical treatment of Dallas sewage, including all of the commonly used chemicals over a pH range from 2 to 12. Ferric chloride or aluminum chloride seemed best with the pH on the acid side for minimum dosage and chemical cost. A miniature plant handling .23 m.g.d. was constructed using chlorine and iron filings as a source of ferric coagulant. Only 1/3 of the chlorine used appeared as ferric chloride generated, a large part of the remainder being converted to free hydrochloric acid which assisted in the pH adjustment. With a B.O.D. of 350 p.p.m., 700 to 1500 pounds of chlorine and 200 to 400 pounds of iron turnings were required per million gallons to effect 77 per cent B.O.D. removal and 86 per cent suspended matter removal. The effluent was clear and the process odorless, but the cost of the proc-

ess is high, probably between $25 and $50 per million gallons of sewage treated.

R. W. KEHR

SEWER CONSTRUCTION IN OKLAHOMA CITY UNDER FEDERAL RELIEF

BY VIRDEN A. RITTGERS

Public Works, **66**, 20 (April, 1935)

Practically all lateral sewers were 8″ diameter, about half clay tile and half concrete, jointed usually with a hot mixture of 40 per cent asphaltic cement and 60 per cent limestone dust. Number 2 clay tile was used in some poorer districts to hold down costs. Every other joint was usually poured before placing the pipe in the trench and 6″ wyes were placed for each building or building location, a riser being run to within 7′ of the surface on deeper cuts. "In general, the additional cost items of overhead, profit, and warrant or bond discounts on contract work just about offset the increased cost of hand labor using relief workers on the force account projects."

R. W. KEHR

THE CHICAGO COMBINATION AERATOR AND CLARIFIER. RESULTS OF TESTS OF PERFORMANCE AT MONROE, WISCONSIN

BY L. H. KESSLER AND P. H. MORGAN

Water Works and Sewerage, **82**, 47 (February, 1935)

A mechanical aerator combined with a settling tank recently developed by the Chicago Pump Company was installed by the manufacturer in an empty tank at Monroe, Wisconsin, for experimental tests under a co-operative agreement with the University of Wisconsin.

The aerator is described as follows: A 12 foot circular steel ring is suspended from beams across the top of the tank. This ring supports the stationary draft tube 36 inches in diameter at the base and 18 inches in diameter at the throat. Above the throat the tube flares slightly to a point 3 inches above the normal sewage surface. A water-tight splash-plate supported by the 12 foot circular ring surrounds the flare. The splash-plate is divided into 12 sections by fins 5 inches high. These fins extend radially for a short distance, and then become tangent to other fins on an arc extending to the edge of the splash-plate. Where the two sets of fins join, one-inch angle weirs are brazed to the splash-plate be-

tween all fins. The sewage lifting device is a three-vane weedless propeller set in the throat of the draft tube about 12 inches below the sewage surface and direct-connected to a vertical induction motor. The propeller shaft passes through a stationary diffuser cone set above the flare section of the tube. This cone has eight deflecting fins so placed as to deflect the sewage outward to the splash-plate in a direction opposite to that of propeller rotation.

Later, in order to provide a complete unit suitable for industries and small municipalities, the equivalent of a final settling tank was installed under the splash-plate. This consisted of two circular sheet metal baffle walls with a 6-inch annular space between and concentric with the draft tube. The outer wall extended down as a conical hopper to the level of the draft tube inlet where a 2-inch annular space was provided to allow for sludge return. Four vertical baffles about 6 feet long were placed between the inside and outside cylinder walls to prevent the rotary motion of the mixed liquor in the aeration section being transferred to the settling chamber. A circular effluent launder surrounded the throat of the draft tube for withdrawing the final effluent.

Operating results show a removal of suspended solids varying from 65 to 87 per cent, and a reduction in 5-day B.O.D. of from 50 to 93 per cent. Except for one run when settled sewage was used, the settling rate of the floc was always below 55 per cent in one hour, and the settling tank had no difficulty in clarifying the mixed liquor. Suspended solids in the mixed liquor varied between 1000 and 2400 p.p.m. The aeration period varied between 7.1 and 28.3 hours and the power consumption between 780 and 3900 Kw. hours per million gallons of sewage.

R. S. Smith

THE WASTE DISPOSAL PLANT OF CANAJOHARIE, N. Y.

By Henry W. Taylor

Water Works and Sewerage, **82**, 79 (March, 1935)

The combined refuse and garbage incineration and sewage treatment plant of Canajoharie, N. Y., was opened December 15, 1934. The plant was built over an existing outfall sewer and provides plain sedimentation with separate sludge digestion for the sewage. The sewage entering the screen chamber and suction well is pumped to the twin settling tanks which are 60 ft. long and 12 ft. wide. These tanks are provided with Jeffery sludge collectors and multiple effluent headers. These effluent headers are placed at the third point, middle of the tank, six feet from and at the end wall and are so arranged that effluent can be withdrawn

through any or all of them. Preliminary determinations indicate that the effluents from all the headers are practically uniform. The sludge is removed from the settling tanks by gravity to an ejector pot provided with a supernatant liquor drawoff. This resettled sludge is ejected to a steel digestion tank with a floating cover housed in a separate room in the plant building. This room has no doors or windows leading to the outside but has two turbine roof ventilators and serves to insulate the digestion tank.

The incinerator, a Morse-Boulger type with a capacity of 18 tons per 24 hours, and the charging room is in one end of the building. Headers and pipe coils are placed in the combustion chamber and form the heating element of a hot water system serving certain rooms of the building and the sludge digestion tank. As the incinerator will be used only 8 hours a day a hot water storage tank was included in the design. This unit serves as a heat reservoir to provide heat during the hours the incinerator is not in service. To avoid excessive temperatures in the coils of the digestion tanks, the water circulated through these coils is controlled by air-operated thermal control valves. The hot water system is also provided with a coal-fired heater and a gas-fired heater in parallel with the heating coils in the incinerator. The incinerator is provided with two gas burners, one adjacent to the primary hearth and one near the secondary hearth. Thus the incinerator provides heat for warming the digestion tank and the sludge gas can be used either in the incinerator to maintain high temperatures and complete combustion or in the gas-fired water heater. The incinerator also disposes of wet screenings from the sewage and will finally dispose of the air dried sludge from the drying beds if desirable. Oil wastes from garages in Canajoharie are collected and brought to the plant where provision is made for filtering and storing them for use as auxiliary fuel in the incinerator. The plant is unusually compact and both processes are easily operated by one man.

R. S. Smith

SEWER LINING MATERIALS TESTED FOR CORROSION RESISTANCE

By H. G. Smith

Engineering News-Record, 114, 444 (March 28, 1935)

Experiments have been made recently on 150 different materials to find a corrosion-resisting lining for Los Angeles sewers which present severe conditions due to direct flows as long as 45 miles. Asphaltic

coatings have been effective for as long as 5 years but shrink and crack when their volatile oils escape. One-eighth inch impregnation with hot paraffin disintegrated under very severe conditions but resisted moderate conditions. The best results were obtained by using clay or hard burned brick with less than 10 per cent absorption and a poured sulphur silicate joint. Experiments are being made using a cast sulphur silicate liner block and a poured sulphur silicate joint, making the lining continuous by fusing the joints with a hot iron. Other experiments are being made using a spray coating of sulphur silicate 1/16 to 1/32 inch thick.

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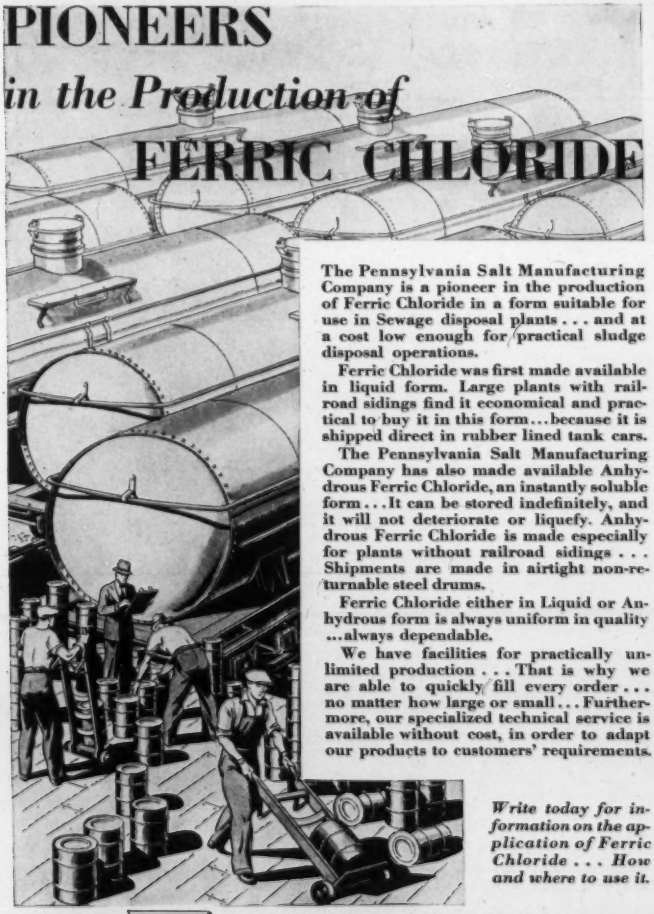

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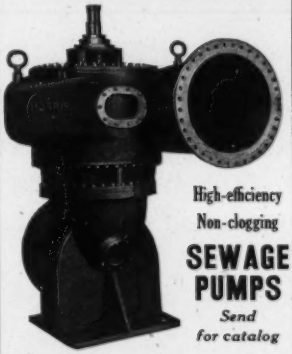

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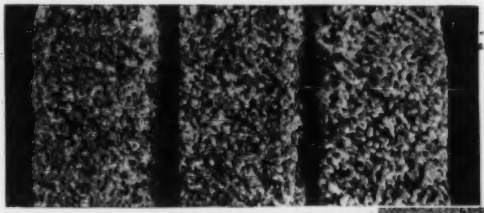

LEFT: Magnified sectional view of three Norton Porous Plates made of grains that are alike in size but differently spaced to give different permeabilities.

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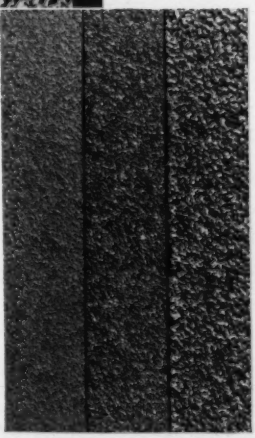

ABOVE: Three Norton Porous Plates whose difference in permeability is obtained by varying the size of the grains of fused alumina.

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Fig. 2050

Fig. 2061

Fig. 1776

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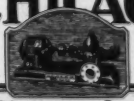

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